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Introduction to Coastal Engineering

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Introduction Coastal Engineering

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1. INTRODUCTION

1.1 The coast

If you were to ask Dutch Coastal Engineers to define "the coast", what would be the reply? Moreover, what would a Chinese colleague answer, if you asked the same question? If these Coastal Engineers had read this book properly, they would answer: "Why do you need the definition?" The simple answer is that there is no absolute definition of the coast and the coastal zone. The area involved depends on the physical and social aspects of the case in question, while, in different countries, different definitions may be used. For example: are river mouths included? The social and natural environment in which the coast is situated also characterizes it so that in every specific case, one must determine what definition of the coastal zone is best. In some countries, the coastal zone is narrowly defined as the area between the High Water line and the Low Water line. However, another wider definition is equally possible; for instance the area between the location where waves start to "feel" the bottom and the most landward side where tidal influence is noticed. In some cases, one takes the area between the –10m and the +10m contour lines with reference to Mean Sea Level. Essential features of the coastal zone include the interaction of the marine environment with the land environment, saline and fresh water, marine and riverine sediments. This creates a region with a unique and wide variety of species and with tremendous opportunities for man.

In general, a coastal zone has a number of (often-conflicting) functions. Among those functions are very important ones: housing, fishing, agriculture, water supply, navigation, nature, recreation (social well being). In the Dutch case, the most important function of the dune coast is probably the defence of the hinterland against inundation. In addition to this, the recreational beach is an example of one far-developed function of the Dutch coast. These two functions are already in conflict with each other. The other functions will further complicate the situation.

Let's take a closer look at this coastal zone, however we may have defined it. The coastal zone system can be schematized in different ways. The system elements can be grouped into two subsystems: the natural and the artificial. The latter is created by human interference, characterized by infrastructure and socio-economic functions. It is the subject of what we generally call Coastal Engineering. The natural subsystem is everything else. It is not hard to imagine that the two subsystems have strong interactive links, and that a proper understanding of the natural subsystem is required by every coastal engineer.

It is also necessary to mention the necessity for conscious coastal zone management. It is predicted (World Coast Conference '93 [1994]) that more than half of the human population of the world will soon be living in the coastal zone (coastal zone in a rather broad sense in this case). Most of the largest metropolitan areas are located along the coast: Tokyo, Jakarta, Shanghai, Hong Kong, Bangkok, Calcutta, Bombay, New York, Buenos Aires, Los Angeles. A lack of balance between the natural processes and the human society in the coastal zone can lead to great poverty, pollution, social problems and structural deficiencies. In short: the world's future depends largely on the future of the coastal zones.

1.2 Coastal Engineering

Coastal engineering is the general term for all engineering activities related to the coast. Typical engineering activities are:

- system, process and problem analysis
- management of information and data acquisition programs
- system schematization and modelling
- planning, design and construction of artificial structures
- measures for the preservation of the natural system

In order to determine which engineering activities might in serve a given situation, all relevant aspects of the coastal system must be studied. Coastal processes have already been divided into natural (for example, sediment transport) and socio-economic processes (for example, zoning (Dutch: Ruimtelijke Ordening) and economic development of the coastal zone). For coastal engineers, the study of the natural processes is a focal point. The study of the socio-economic processes tends to be included in the study of coastal zone management. This is a multi-disciplinary activity, in which the coastal engineer must play an active role.

As stated above, the coastal zone borders cannot be clearly defined. The coast does not stop where the sea starts. However, where does it end? At the edge of the continental shelf perhaps? Or at the boundaries of one's technical skills? The inland border is even more difficult to determine. A river can influence a coast via the sediment it carries since it can be a sediment source. Any change in the river regime may thus have serious consequences on the coast. Thus, either the whole river basin or at least part of it must be considered as an element of the coastal zone. In other words, the field of play of the coastal engineer covers a variety of subjects.

Engineering activities have an ever-increasing influence on the coast because the tools available to interfere in the coastal processes (i.e. artificial sediment transport by means of dredging) have become very powerful. This means that the need for adequate coastal zone management is also growing rapidly.

Back to the engineering key words. Most of them (problem, information, measurement, model, artificial measures, and structures) need to be viewed in a larger context and this context forms the content of this book.

1.3 Structure of these lecture notes

1.3.1 Position in the curriculum

In order to introduce the reader to the basics of coastal engineering these lecture notes contain a selection of subjects. This means that many things cannot be treated in depth because the practice of coastal engineering is too diverse to encompass every aspect in one short course. However, the main processes that take place around the coast are described. In the curriculum of Delft University, the present introductory course is followed by more specific lectures on morphology, coastal zone management, tidal inlets and estuaries, and on coastal structures. The lecture notes of those courses together provide a more complete and comprehensive view of coastal engineering as a whole. Literature, out of which much information has been put into this book, is listed and recommended warmly.

1.3.2 Contents of this book

First of all, Chapter **2** gives a brief description of the coast as a physical system. As an important basis, the plate tectonics theory is described. This is the terrain of the geology. In addition to this, the situation in the Netherlands is treated. This country has had a long history of engineering works related to the coast. The coastal history of the Netherlands does not start, like history in Dutch schools, with Romans conquering Gaul but some 18,000 years before the present. Much of the bed of the North Sea was land at that time but the sea level started rising and brought the coastline nearer to what is now the Dutch coast. Many changes were documented during the historical period and now the works of the Delta project and many other equally visible and less visible aspects illustrate coastal engineering practice.

Climatology, oceanography and morphology are the names under which the natural processes can be defined and together they form the complex system of natural processes that give shape to the coast which are described in Chapters 3 through 5.

Chapter 6 is devoted to coastal formations. Different parts of the world are visited to give more detailed information about the dynamics of different coastal types.

Chapter 7 deals with the cultural aspects of the coastal system, as far as they are relevant to engineering practice and most specifically to the social and economic aspects. To man, the coast has always been very attractive. Socio-economic activities have always been intense in the coastal zone, and they are still growing. Therefore, global socio-economic problems, like poverty, are serious in the coastal zone, too. The answer to them is commonly thought to lie in Integrated Coastal Zone Management. An introduction to this form of management is given.

Where fresh and saline water meet, problems caused by differences in the density of the water can be expected. Another aspect of the coastal zone is its vulnerability to pollution. Chapters 8 and 9 are dedicated to both types of problems.

Chapter 10 of this introduction into coastal engineering gives some practical details of the subject. Several problems which might be encountered in the everyday practice of the coastal engineer have been added. Design skills form a major part of this practice and therefore attention paid to some coastal design basics.

Chapters 11 to 15 describe the basic aspects of dredging.

Lastly, there are six appendices with additional information on the history of the planet earth, the mechanics of relative motion (Coriolis acceleration), the two largest coastal engineering works in the Netherlands (The reclamation of the "Zuiderzee works" and the Delta Project), the reading of hydrographic charts and on centrifugal pumps.

1.4 Authors

This book has been compiled by a great number of people, on the staff of or attached to the Section of Hydraulic Engineering of the Faculty of Civil Engineering and Geosciences of Delft University of Technology.

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Many others contributed by correcting text or preparing figures. In this respect the contribution of the following persons is greatly acknowledged:
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1.5 Miscellaneous

This book has been written by authors from the Netherlands. Since, some of the techniques discussed were developed in the Netherlands centuries ago the English sometimes has a distinctly Dutch flavour, for which the authors make no apology!

To avoid misunderstandings, the reader is referred to a seven-language vocabulary on coastal engineering (The Liverpool Thessaloniki Network [1996]).

In this book, the metric (mks) system (based on the definition of mass [kg], length [m], and time [s]) is used, except in the case of some widely-accepted nautical and hydrographic terms such as knots, fathoms and miles.
Where applicable:

- the X-co-ordinate is used in the direction of current or wave propagation
- the Y-co-ordinate is used for the horizontal direction, normal to the X-co-ordinate
- the Z-co-ordinate is defined in the vertical direction, positive upward, with the origin either at the bottom or at the surface level

Since data from existing literature, often from different disciplines, are used and copied, the reader cannot expect a 100% systematic use of symbols throughout the book. For instance, the water depth may be indicated by the letters h, d or D, stone size by d or D. The letter σ may thus be used for the stress in a material, or as a symbol for the standard deviation. Therefore, the list of symbols must be used with care since it is not unambiguous. If serious confusion could arise, the symbols will be defined and explained where and as they are used. The authors feel that students should be trained to adapt to different notations when they read literature from different sources.

2. FROM GENESIS OF THE UNIVERSE TO PRESENT-DAY COASTLINE

2.1 Introduction

How were the present coastlines of the world formed? Any single coast is the result of processes operating over a number of widely varying time scales:

- the slow geological processes of mountain formation and erosion that require millions of years
- the gradual sea level changes requiring thousands of years
- the annual and biannual morphological variations due to fluctuations of the climate
- superimposed on these, the day-to-day actions of the wind, waves, currents, and tides

For a relatively short period, there has also been the influence of humans. Originally, people were causing little more than scratches on the world map. The result of modern construction equipment is that human influence on the coastal forms is even visible from space. The reaction time of the natural system to large-scale projects like the closure of the Zuiderzee and the Delta Project is in the order of decades (50 to 100 years).

In the same way as the time scales differ widely, the spatial scales of the processes differ considerably. We have to distinguish:

- the interaction between wave and sand grain in the breaker zone (on a scale of metres)
- the morphological effects of longshore and cross shore transport (on a scale of kilometres)
- the effect of glacial periods and associated sea level variations (on a scale of hundreds of km)
- the effect of drifting continents (on a scale up to 10 000 km)

To understand the present image of the world coasts, it is necessary to start briefly at the genesis of the universe, to discuss some major events in geological history, and to focus on the most recent geological history of the earth. Finally, the general review will be illustrated by a more detailed description of the recent history of the Dutch coast.

2.2 Genesis of the universe, earth, ocean, and atmosphere

When we discuss the genesis of the universe, our solar system, the creation of the earth, we touch issues that are subject to more than a single interpretation. In this book, we will follow the theories that are supported by most scientists. The oldest history of the universe belongs to the world of the astronomy, a science that is based on the observation of events that are still taking place around us, or rather that took place around us, since the distances are so great that we count the delay between event and observation in terms of light years. This field of science is marked by a very rapid development because very sophisticated instruments are now at our disposal owing to the start the space exploration.

It is widely accepted that the universe originated in a great explosion, the so-called 'big bang'. This model is consistent with observations first made in 1929 that distant galaxies are receding from the earth at velocities proportional to their distance from earth. In 1948 George Gamow predicted that astronomers would one day detect background microwave radiation left over from the big bang. In 1965, Penzias and Wilson proved Gamow right when they detected this radiation, and subsequent measurements provided further confirmation. Other theoretical models have been proposed to explain the origin of the universe, but these have proved deficient when tested against observations and physical measurements.

From observations and calculations it has been deduced that it took about 15 billion years from the big bang to the formation of our solar system with the planets, as we know them at present. That sets the start of the history of "planet earth" in a stricter sense at about 4 to 5 billion years before the present. In Table 2-1, the chronology is shown from the big bang to the creation of our planet. For those who are interested in the formation of our solar system, Appendix 1 gives additional information.

Event	Time before present
Big Bang	20 billion years
Particle creation	20 billion years
Universe becomes matter dominated	20 billion years
Universe becomes transparent	19.7 billion years
Galaxy formation begins	18-19 billion years
Galaxy clustering begins	17 billion years
Our proto-galaxy collapses	16 billion years
First stars form	15.9 billion years
Our parent interstellar cloud forms	4.8 billion years
Proto-solar nebula collapses	4.7 billion years
Planets form; rock solidifies	4.6 billion years

Table 2-1 From the 'big bang' to formation of planets (1 billion = 10^9)

2.3 The geology of planet earth

When we describe the physical history of planet earth since it came into existence some 4 to 5 billion years ago, we leave astronomy and we enter the realms of geology. Geologists do not go back to the big bang in their descriptions. They have their own way of subdividing the past time into eras, periods, and epochs. For many years this subdivision was based on the interpretation of samples from the earth crust and on the presence of characteristic fossils. Before the mid-20th century, the only available technique was fossil time-scaling. This technique only gave information about the relative age of certain layers, compared to each other. Radiometric dating is a relatively new technique used to determine the age of units. This modern technology, based on the decay of radioactivity in certain isotopes, yields a more accurate result. It must be kept in mind, however that the decay expressed in isotopic years (for instance C14 years[1]) is not always exactly equal to the age in sun-years.

In this way, Geology has produced a time scale that covers the roughly 4 to 5 billion years since the formation of the planet. This time scale is divided in some main periods and subdivided in a much larger number of sub-periods. The more recent periods are described in far greater detail than the initial stages of development. When looking at the geological time scale, one should realise that the sub-periods that can be distinguished become ever shorter. Table 2-2 shows the geological time scale.

An example of the significance of fossils in dating geological events is the boundary between the Mesozoic ("interval of middle life") and the Cenozoic ("interval of modern life") eras. It is marked by the disappearance of hundreds of species, including the dinosaurs, followed by the appearance or sudden proliferation of many new species (Stanley [1986]). The Cenozoic is subdivided into the Tertiary and the Quaternary. The Quaternary consists of the Pleistocene and the Holocene.

The epochs of most concern to coastal engineers are the Pleistocene and Recent or Holocene, extending back a total of 1.8 million years before present. During the Pleistocene, pronounced climatic fluctuations occurred. Continental glaciers periodically covered vast areas of the

1 C^{14} is a radioactive carbon isotope, which originates from Nitrogen-14. Cosmic radiation causes the production of C^{14} in the atmosphere. This radioactive isotope is incorporated in the biosphere. It decays with a half-life time of 5730 years. It is mainly used to date the younger geological processes (of the past 50,000 years)

continents in what is called the modern Ice Age. Today many of the geomorphologic features shaped or deposited at that time are still clearly recognisable. From around 18 to 15 thousand years ago, the global climate was warming again. At the same time the Holocene Transgression started with the beginning of global rise in sea level. Many morphological features associated with the coastal environment are Holocene in age, but the pre-existing geology is often visible, as well. This most recent history of the earth also contains evidence of primitive human life. It reflects the dependence of mankind on global changes.

Within the period encompassed by geological history, two aspects deserve special attention in lectures about coastal engineering. They are the process of continental drift (plate tectonics) and the relative changes in sea level.

Plate tectonics: the changing map of the earth

The theory of plate tectonics has a complicated history that reaches back to the global maps created after the great ocean voyages of the 16th and 17th centuries. As the maps became more accurate, the landmasses took on the appearance of pieces of a giant puzzle. Sir Francis Bacon is credited as the first to note this resemblance: in 1620 he wrote that the coastlines of South America and Africa would fit together perfectly if the ocean were not between them.

Remarks	Main period	1st sub-division	2nd sub-division	3rd sub-division		Time scale in years before present (BP)
	Fanerozoic	Cenozoic	Quarternary	Holocene (or old name: Alluvium)	Sub-Atlanticum	2 900 BP
					Sub-Boreal	
					Atlanticum	
					Boreal	
					Pre-Boreal	10 000 BP
				Pleistocene (or old name: Diluvium) (Glacial Periods)	Weichselian glacial age	
					Eemian	
					Soalian glacial age	
					Holsteinian	
					Elsterian glacial age	
					Cromerian	
					Menapian glacial age	
					Waalian	
					Eburonian glacial age	
					Tiglian	
						$1.8 * 10^6$ BP
			Tertiary	Pliocene		
				Miocene		
				Oligocene		
Africa, America separated				Eocene		
End of dinosaurs				Paleocene		$65 * 10^6$ BP
Abundant life		Mesozoic	Cretaceous	Late Cretaceous		
				Early Cretaceous		
			Jurassic	Malm		
				Dogger		
				Lias		
			Triassic	Keuper		
				Muschelchalk		
				Bont sand stone		$225 * 10^6$ BP
		Paleozoic	Permian	Zechstein		
				Rotliegendes		
			Carboniferous	Silesian		
Start of reptiles				Dinantian		
			Devonian	Late Devonian		
				Middle Devonian		
				Early Devonian		
			Silurian			
			Ordovician			
			Cambrian			$600 * 10^6$ BP
	Cryptozoic		Precambrian			
First primitive life						$3.2 * 10^9$ BP
Formation of planet						$4.75 * 10^9$ BP

Table 2-2 Geological time scale

In 1912 Alfred Lothar Wegener presented a comprehensive scheme to explain the distribution of the continental landmasses. He believed that the continents had slowly drifted apart from a primordial super-continent, which he called Pangaea (Greek for "all earth"). He envisioned a single world ocean, Panthalassa ("all ocean"), with a shallow sea, Tethys (from Greek mythology, the mother of all oceans), located between Laurasia and Gondwanaland, the northern and southern portions of the super-continent (Figure 2-1). Using accepted geologic and paleontological data, Wegener provided good supporting evidence for the continuity of geologic features across the now widely separated continents. Three years later, Wegener produced his major work, "Die Entstehung der Kontinente und Ozeane", in which he presented an enormous amount of evidence in support of his theory.

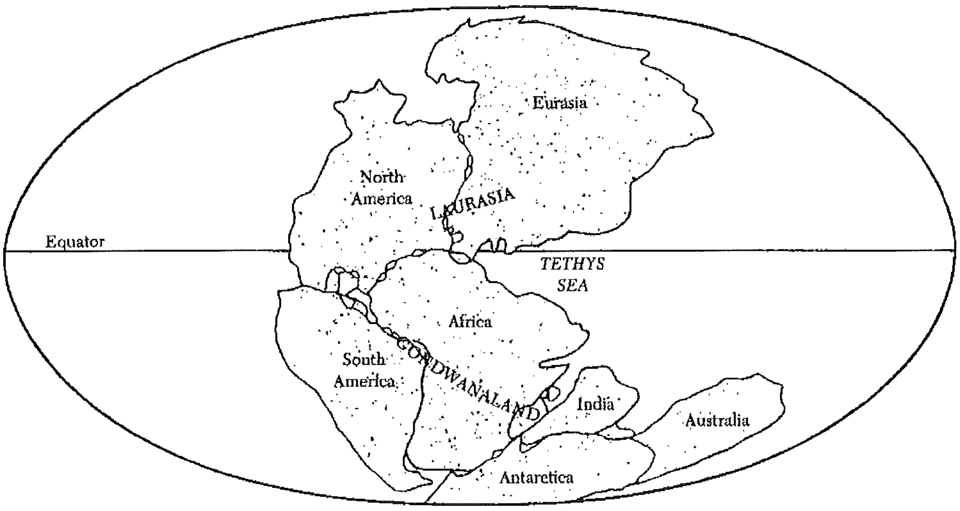

Figure 2-1 Continental landmasses during the early Trias (Davis, 1994)

The continental landmasses that formed Pangaea gradually drifted from their original positions. In Figure 2-2 this process is illustrated. They reached intermediate locations 135 million years ago, between the Jurassic and Cretaceous Periods. After almost 200 million years, the continents reached their present positions, though we can observe that they are still drifting! Nowadays, it is known that even before the formation of Pangea, the continents were already drifting. Even before the start of the Permian, a proto-Atlantic Ocean existed. Closure of this ocean caused mountain formation in Scotland, Norway and North America.

Plate tectonic theory states that the continents, being part of the lithosphere, the Earth's uppermost layer containing the crust, drift on the semi-molten underlying material we call the asthenosphere, or the upper mantle. By the 1960's, scientists had concluded that the lithosphere is divided into 12 large, tightly fitting plates and several small ones. Six of the large plates bear the continents; the other six are oceanic. And, as Wegener asserted, all of the plates are in motion (Figure 2-3).

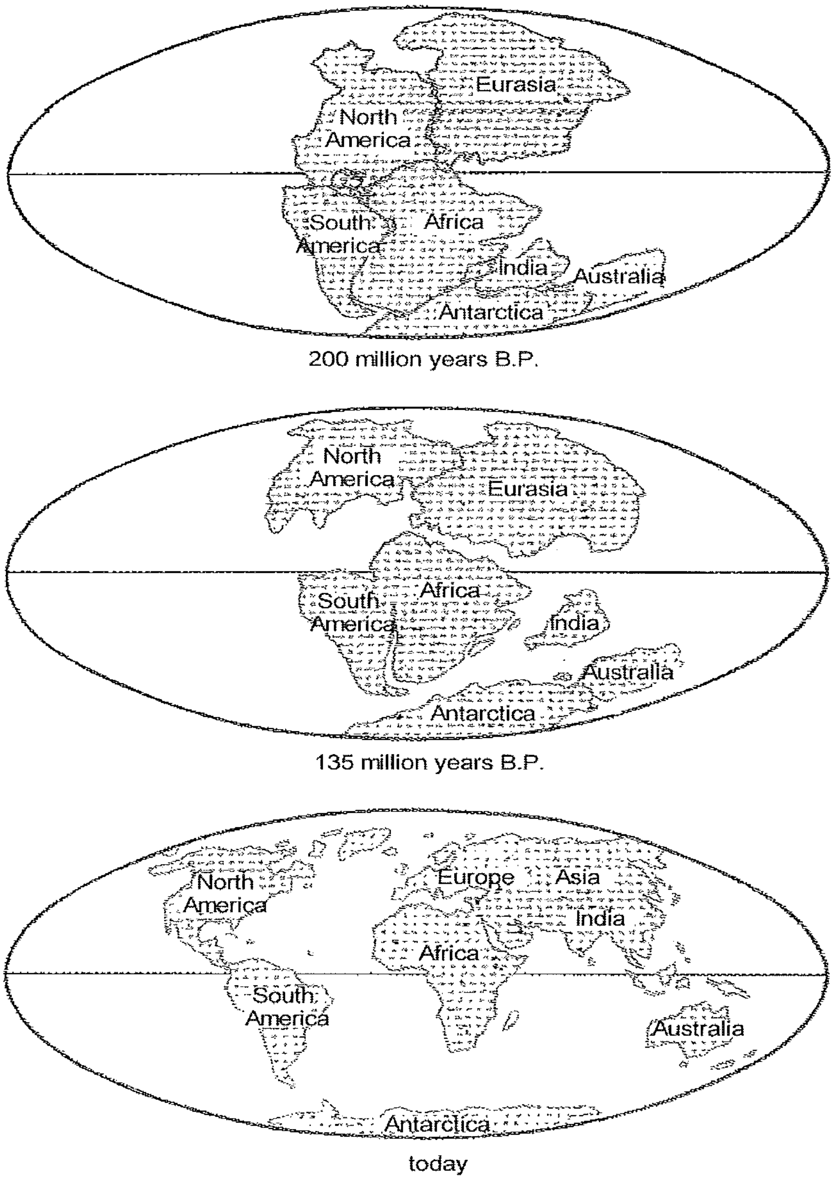

Figure 2-2 Continental drift (Wegener, 1924)

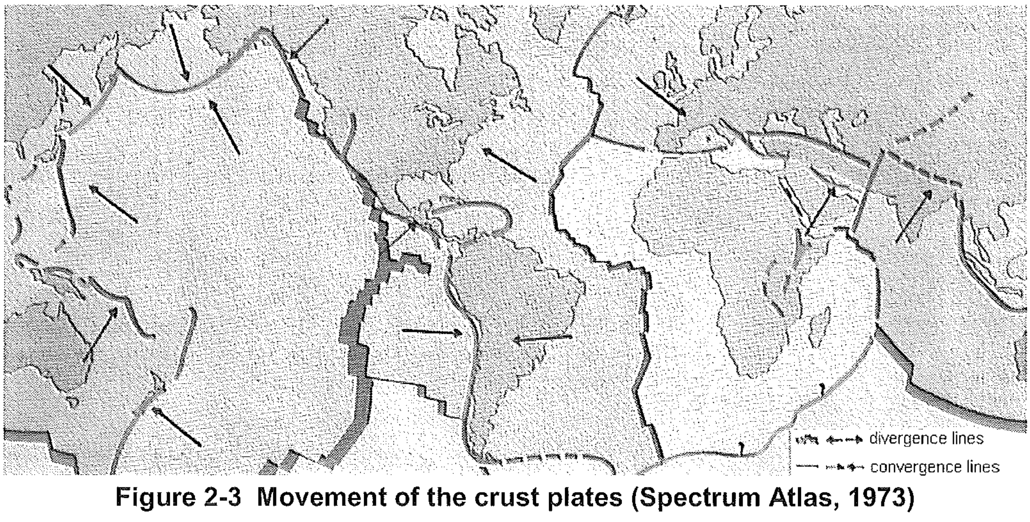

Figure 2-3 Movement of the crust plates (Spectrum Atlas, 1973)

Correlated to the process of plate drift, at certain places, the semi-molten asthenosphere material can be driven to the earth surface. This happens in the so-called oceanic ridges. Following from that, new (oceanic) earth crust is being formed (Figure 2-4). This process is associated with divergence. The age of the crust on both sides of the mid-ocean ridges increases with distance. The oldest crust is found in the trenches (Figure 2-4). Therefore, to geologists, the characteristics of the sea bottom reveal information about earth history.

At other places, instead of divergence there is convergence. In the oceanic trenches, one plate dives under the other. At those places the earth crust is returning to the asthenosphere and partly melting again. This process of convergence is often accompanied by seismic and volcanic activity.

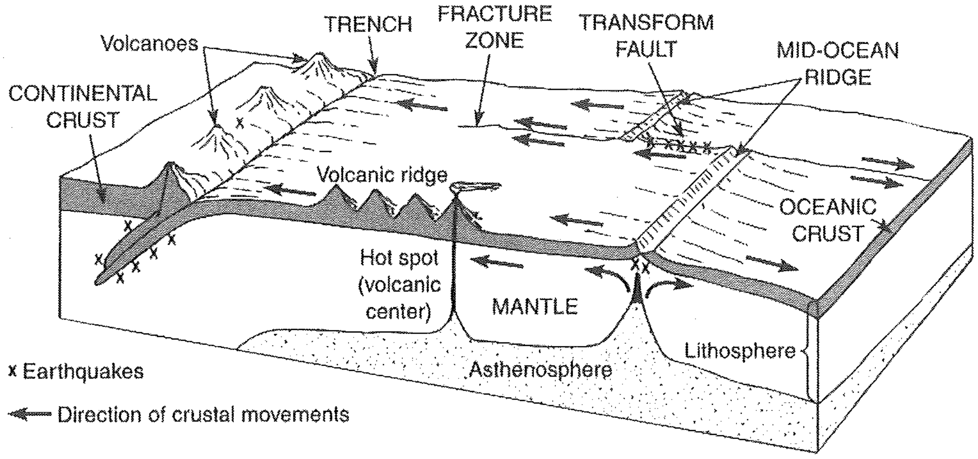

Figure 2-4 Movement in the earth's crust

Since Wegener published his theory, many years of debate and research have passed. Only during the last three decades, has proof for the tectonic movement of plates been found. This proof has been provided by:

- the Ocean Drilling Programme (ODP)
- the worldwide installation of sensitive instruments for the detection of nuclear tests
- satellite observations of the earth

The Ocean Drilling Programme consists of basic research into the history of the ocean basins and the nature of the crust beneath the ocean floor. Many countries take part in this project, and it is still in progress. Special drilling equipment is used in order to take samples of the ocean floor from great depths (up to 9 kilometres below the water surface). Hundreds of boreholes have been drilled in an internationally co-ordinated research project.

In the Ocean Drilling Programme, the top layers of bottom sediment are examined with respect to their origin. In this way, plate velocities can be estimated. Secondly, fossils found in those layers can tell their story of temperature change. One of the things that makes the ocean bed so interesting is that it provides a way to determine a continuous earth history by means of a relatively thin bottom layer. Usually, the ancient continental masses are covered by huge quantities of sediment, which are precipitated during relatively short periods.

In connection with the test ban treaty for nuclear weapons, accurate sensors have been installed around the globe to detect nuclear blasts. These instruments also detect earthquakes. From the observations, it appears that frequent seismic activity takes place along the edges of the moving plates. In this way, the boundaries of the plates could also be detected.

Nowadays the rates of plate movement are also measured with the aid of satellites that use very accurate geodetic positioning systems (like DGPS). The movements appear to vary from about 1 cm a year at the Mid-Atlantic ridge to 10 cm a year at the East Pacific rises in the south-eastern Pacific. Last but not least, the mid-Atlantic rift becomes visible at the surface on Iceland in a very spectacular way (Figure 2-5).

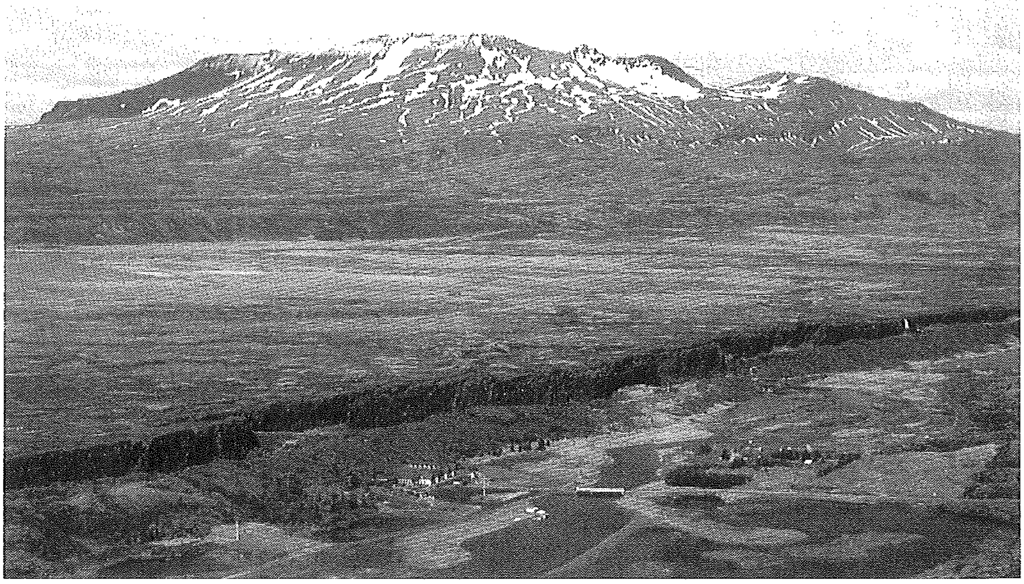

Figure 2-5 Mid-Atlantic Rift at Thingviller National Park, Iceland

Sea level changes

Since it was formed, the ocean has never been constant or static. The process of "new water formation" by volcanic activity (see Appendix 1) produced very small water level changes. But there have been, and there still are, other processes, which have a much stronger effect on the global sea level. The most important of these is certainly temperature change. If the global temperature rises, this leads to the melting of ice caps and expansion of the total water mass. This has happened often during earth's history, and it is still happening. Melting of the ice caps also reduces the load on the crust as a result of which areas previously covered by ice are rising, whereas other adjacent areas that were never covered with ice tend to sink due to the effect of isostacy (equilibrium flotation) on the earth crust. In this way, sea level changes are relative movements; they can be caused by absolute changes of the sea level and/or by absolute movements of the continental crust.

Sea level changes can affect the coastal zone very strongly. Sea level changes are relative movements and because of their nature, they vary from place to place. As the shoreline moves, it either exposes or inundates coastal areas and, in so doing, causes the character of the coast to change. Additionally, the position of the shoreline influences coastal processes that shape the coastal environments. Inundation is also called transgression, whereas drying of the land is referred to as regression.

The following processes (Davis, 1994) cause sea level changes in the present day conditions:

1 tectonic activity
2 climatic fluctuations (natural or man made)
3 regional subsidence due to compaction and fluid withdrawal
4 subsidence and rebound of the lithosphere
5 changes in the volume of the world ocean
6 advance and retreat of ice sheets
7 continental rebound

In order to understand these processes, the geological, climatic, oceanographic and morphological influences on coastlines must be described. The geological processes will be explained in the following sections of this chapter. The other factors will be treated in subsequent chapters.

Rising sea levels are a danger to the people in many countries and since coastal defence is an expensive business, the poor countries are the most vulnerable to this danger.

2.4 Tectonic classification of coasts

Coasts are strongly influenced by plate tectonics. If a coast is situated close to a plate boundary, it develops differently from a coast that is not. Inman and Nordstrom (1971) made a classification of coasts, which divides all the continental coasts into three major types:

- leading edge or collision coasts (those associated with the leading edge of a crustal plate)
- trailing edge coasts (those associated with the trailing edge of a plate)
- marginal sea coasts (those bordering a sea enclosed between the landmass and a volcanic island arc at the plate boundary)

Island arc coasts are not considered in this classification.

The formation of the first two types, the leading edge and the trailing edge coast, is drawn in Figure 2-6.

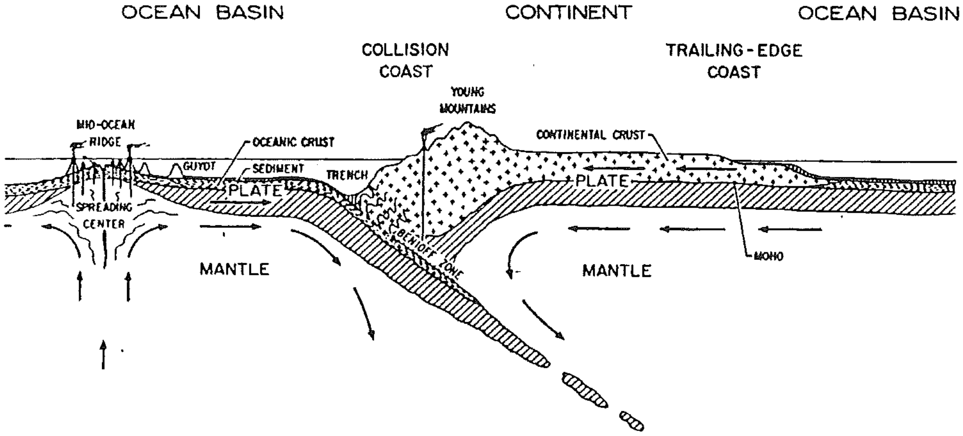

Figure 2-6 Formation of leading and trailing edge coasts

- Leading edge coasts (or collision coasts) develop along the border of a landmass where the oceanic edge of one plate converges with the continental edge of another. They are distinguished by rugged, cliffed shorelines. The convergence between the two plates may produce subduction zones as the denser oceanic plate descends beneath the continental edge of the other plate. The tremendous friction created by the converging plate edges causes the lighter continental crust to fold and buckle, creating the mountain ranges that can often be found near leading edge coasts. Moreover, the stresses that develop during the subduction process cause earthquakes. The West Coast of the American continent is a good example of a leading edge coast. The Andes Mountains are the result of the buckling process. This is illustrated by Figure 2-7, the coast near Antofagasta, Chile. The rising magma may also create volcanic activity, which is the case in the Andes and the mountain ranges along the W. coast of N. America.

Other examples of leading edge coasts are found along the coast of Malaysia and Sumatra, in Turkey and Greece and in Japan and New Guinea. This can be noted from Figure 2-3!

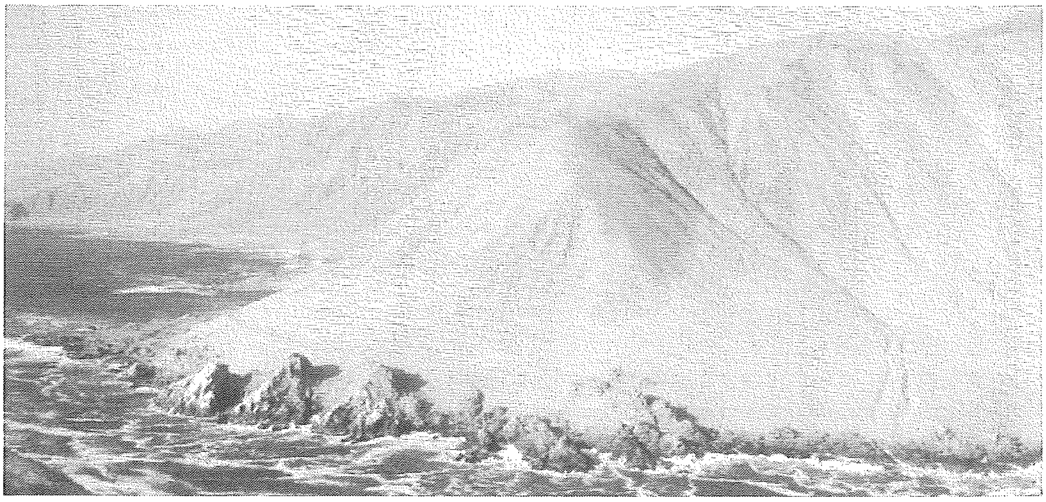

Figure 2-7 Leading edge coast near Antofagasta (Chile) After Davis (1994)

The steep mountain slopes of leading edge coasts hold rapidly flowing streams and small rivers that quickly erode their beds. Because the watershed is at a high elevation near the coast, the rivers are short, steep, and straight. They transport large quantities of sediments directly to the coastal areas, giving no opportunity for sediments to become entrapped in a meander, on a natural levee, or on a flood plain. The rivers deposit their sediment loads into coastal bays or directly onto open beaches.

Even though mountain streams deposit large amounts of sediment on the coast, they do not produce deltas (Davis [1994]). In fact, none of the world's 25 largest deltas occurs on leading edge coasts, because this tectonic setting does not have a shallow, nearshore area on which the sediment can accumulate, and because waves are usually large along leading edge coasts. If sediment eventually does accumulate, it is soon dispersed by the large waves coming from the deep ocean.

- <u>Trailing edge coasts</u> develop in association with a part of the continental lithosphere that is not at the leading edge of a plate (Figure 2-6) and that typically has been tectonically stable for at least tens of millions of years. All these years, erosion processes have taken place and converted hills and cliffs into coastal and submarine plains. Along these coasts, one can find huge deposits of sediment. They are shaped and reshaped by currents, wind and waves. These are the coasts where one finds barrier islands, deltas and other sedimentary shapes. Inman and Nordstrom have categorised trailing edge coasts on the basis of their plate tectonic settings as Neo-trailing edge coasts, Afro-trailing edge coasts, and Amero-trailing edge coasts. The three subtypes refer to details of the erosion process after the breaking up of a landmass.

A Neo-trailing edge coast occurs as plates diverge from an active spreading centre. If the newly produced crust forms a coast, it represents the first stage of coastal development. It is only a few million years old. Coasts like this must have existed just after the proto-Atlantic developed, as the continents (Africa and South America) split up during the Triassic period, 190 million years ago. The coarse gravel beach along a high-relief coast on the Sea of Cortez, Mexico, provides an example of a Neo-trailing edge coast. Its photograph is shown in Figure 2-8.

Figure 2-8 Coarse gravel beach along a high-relief coast on the Sea of Cortez, Mexico

The African continent occupies a position in the middle of a crustal plate that has little tectonic activity along its margins, and has been relatively stable for many millions of years. The typical coastal features of this continent have led to the name Afro-trailing edge coast. Afro-trailing edge coasts have developed pronounced continental shelves and coastal plains, but these features lack the extent of more mature coasts, and sedimentary features such as large deltas are rare. The African continent has been relatively stable for a long time, so no extensive, high mountain ranges are present. The modest to large river systems drain areas of only modest relief, so sediment gets a lot of time to be deposited before arriving in the river mouth.

Amero-trailing coasts, geologically the maturest coastal areas, are represented by the east coasts of North and South America. Both are tectonically stable portions of the continents, well away from the plate boundary, and have been located so for at least several tens of millions of years. The combination of long-term tectonic stability, a moderate climate, and the development of a broad coastal plain has provided huge quantities of sediment to the coastal system. During this time numerous large, meandering river systems have developed. For more than 150 million years, these rivers have been carrying sediment across a gentle incline. As they have deposited sediment at or near their mouths, they have created broad, low relief coastal plains on the landward side and, on the seaward side gently sloping continental shelves. Wave action along Amero-trailing coasts is limited, because the water of the gently sloping inner continental shelf is shallow. Large mid-ocean waves lose energy as they progress across the shelf, and consequently do not inhibit deposition of sediment along the coast. An example is the extensive mangrove swamp and tidal flats, which cover the low relief Amero-trailing edge coast near the mouth of the Amazon River in Brazil, in Figure 2-9.

- Marginal sea coasts are near to the plate boundary where a collision is occurring, but are kept apart from its influence. In these places, a moderate-sized marginal sea separates a passive and tectonically stable continental margin from the volcanic island arc at the plate edge near a subduction zone. Although fairly close to the convergence zone, the marginal seacoast is far enough away to be unaffected by convergence tectonics - it behaves like a trailing edge coast. Well-developed rivers carry large quantities of sediment to the coast, where a broad and gently sloping continental shelf provides an ideal resting-place for large quantities of land-derived sediment.

The restricted size of the marginal sea limits the size of waves that develop. In addition, the gentle slope and shallow waters of the continental shelves in these areas attenuate wave energy. Hence, the combination of relatively low-energy coastal conditions and sizeable sediment loads allows the formation of large deltas and other coastal sedimentary deposits such as tidal flats, marshes, beaches and dunes. The great rivers of southeastern Asia and the Gulf region of the US, both areas of mild climate and abundant rainfall, have deposited their sediment loads on marginal seacoasts to create some of the largest deltas of the world.

Figure 2-9 Coast near the mouth of the Amazon River in Brazil

2.5 The Dutch coast

2.5.1 Geological history of the Dutch coast

The actual history of the present Dutch coast starts from the end of the Pleistocene. At the end of the Pleistocene, some 10,000 years ago, the area presently known as the southern North Sea was completely dry, forming a plain that connected England with the rest of Western Europe. This plain was intersected by the main rivers like Thames, Rhine, Scheldt and Meuse. The end of the Pleistocene and the start of the Holocene were marked by a rise in temperature. The rising temperature caused a considerable rise of the sea level as indicated roughly in Table 2-3, and more in detail in Figure 2-10.

Table 2-3 shows the time schedule for the most recent (Holocene) part of the geological history of the Dutch coast expressed in sun (calendar) years (years AD) and in C-14 years (years BP).

Deposits from the Pleistocene are still visible on the present surface in the eastern part of the Netherlands; the Western part is almost completely covered with the more recent deposits from the Holocene. In the Pleistocene deposits we can still distinguish numerous ice-pushed ridges (often containing moraine material or glacial till).

GEOLOGICAL TIME SCALE	AGE C14	AGE calendar	sea level position (m)	OBSERVATIONS COASTAL DEVELOPMENT
Late Weichselien				
Holocene	10 000			
Prae-Boreal				
Boreal	9 000			8700 BP Sea protrudes into Strait of Dover/Calais 8300 BP Connection between Northern and Southern part of North Sea is formed
Atlanticum	8 000		- 25	7800 BP Average coast line position 25 km West of the present position
	7 000	5500 BC	- 15	Period of coastal retreat Dutch coast develops into open Wadden Coast
	6 000		- 7	
Sub-Boreal	5 000	3850 BC	- 5	5000 BP Start of closure dutch coast, coastal propagation and formation of the old dunes; Coastline on average 8 km East from present position; Old Rhine Mouth is activated
	4 000		- 3.0	
				3200 BP Closure of Bergen inlet
	3 000		- 1.7	
Sub-Atlanticum		1100 BC		Period in which coastal progradation stops and retreat starts again; Old Rhine mouth non-active
	2 000	AD	- 1.0	
				ca. 500 AD Inundations in the SW-part of the Netherlands (Zeeland)
				ca.1100 AD Inundations in the NW-part of the Netherlands (kop van Noord Holland)
	1 000			1000-1600 AD Formation of the young dunes
	0	1999	NAP	

Table 2-3 Time Table with the main events for the Dutch coast during the Holocene

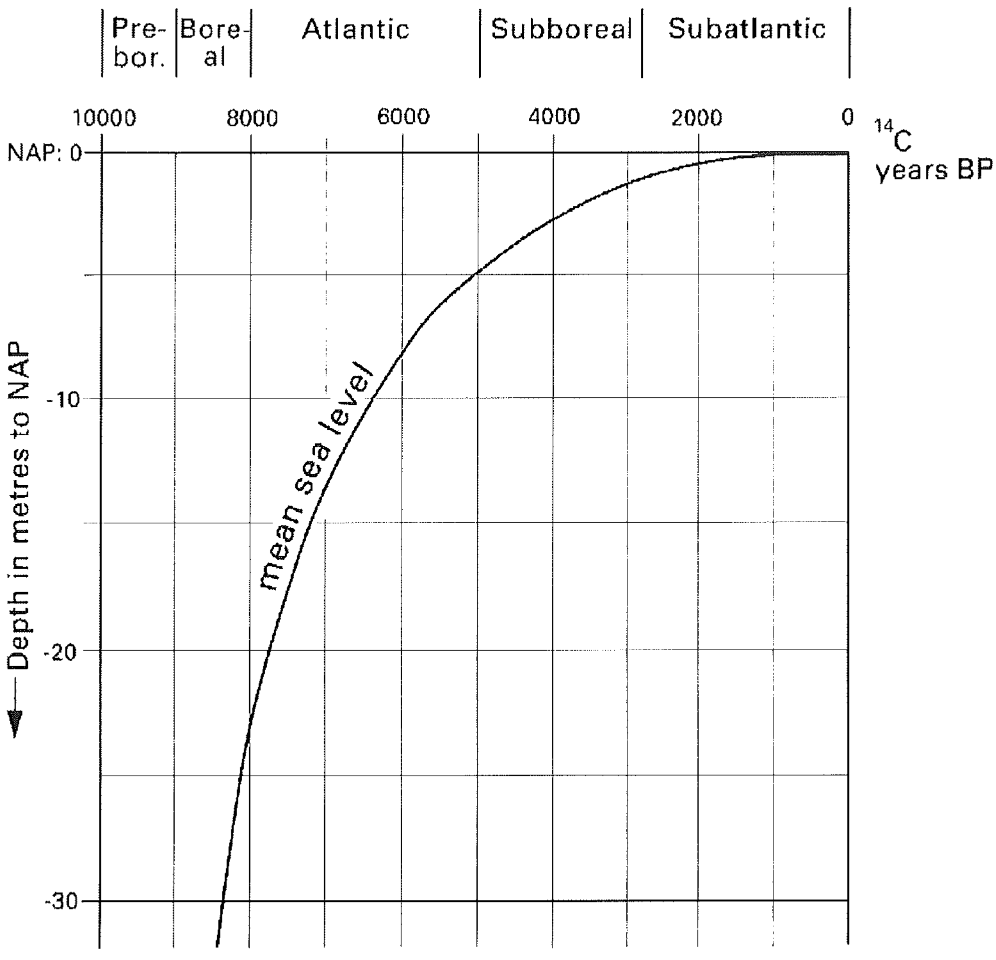

Figure 2-10 Relative Holocene sea level rise along the Dutch coast

In the early years of the Holocene, the sea level rise amounted to 1m/century or more. About 9000 years ago, the plain between England and Europe started to flood. At first, the water came in from the south, through the English Channel. In the meantime, the northern North Sea was formed due south of the retreating polar ice. The separation between the southern and northern parts of the North Sea extended from the present Dutch Wadden Sea to the west. The plain was completely flooded between 8500 BP and 8000 BP. As the icecaps continued to melt, and the land subsided in other places, the sea transgressed further. About 7500 BP, the Dutch coastline was situated some 25 kilometres west of its present position and it migrated continuously in landward direction (see Figure 2-11 and Figure 2-13).

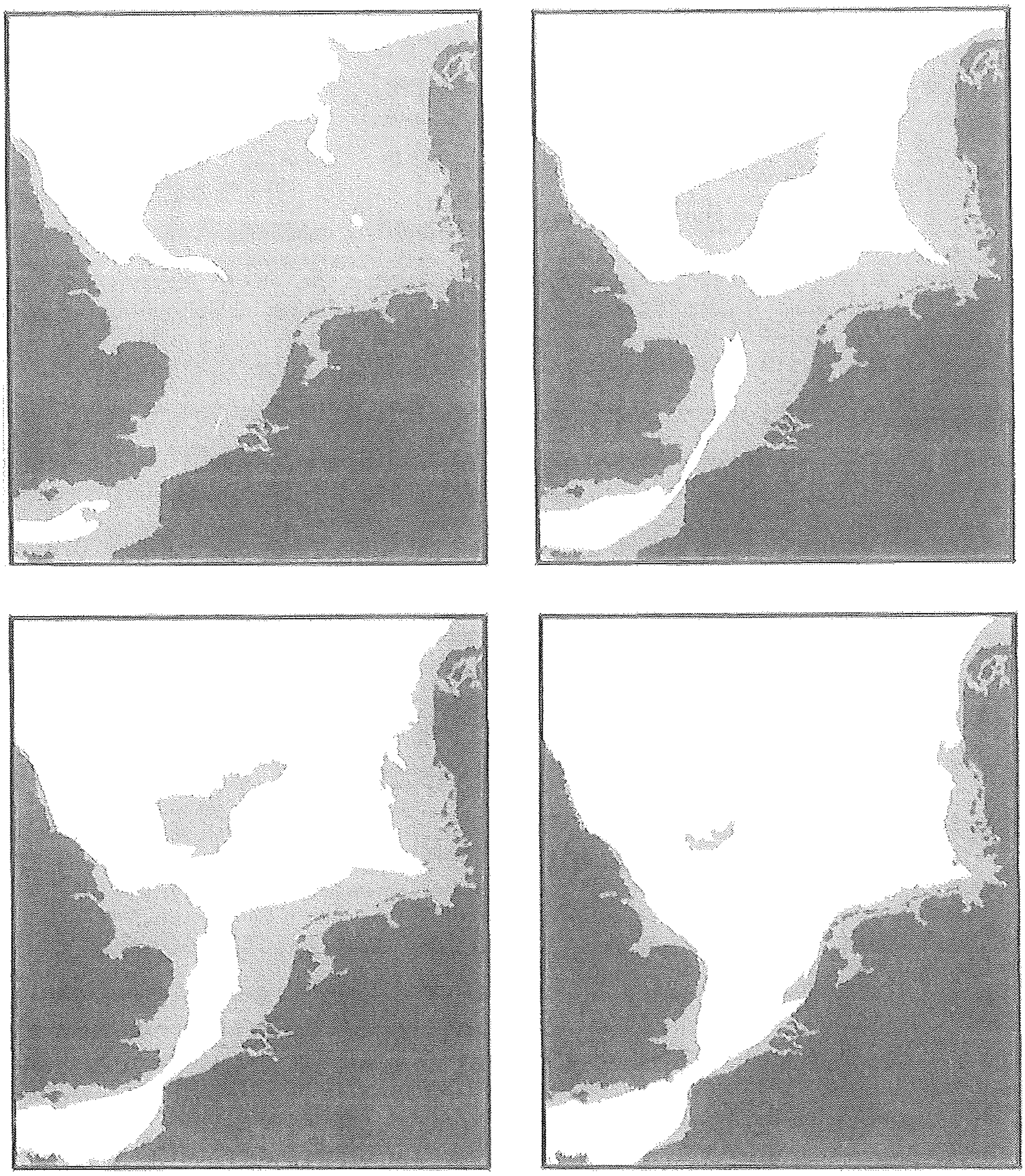

Figure 2-11 Four stages in the formation of the North Sea (after Zagwijn, 1986)

The Pleistocene landscape in what is now the Netherlands had a distinct relief. Generally speaking, the height increased to the east. Two large, east-west trending valleys dissected this landscape: one running from North-Holland, between Alkmaar and Castricum, to the Noordoostpolder, and one further south, approximately at the present position of the rivers Rhine and Meuse (see Figure 2-12). In the northern part of The Netherlands, there were two smaller, SSW-NNE running valleys. The north-western part of the Netherlands had a relatively high elevation due to ice-pushed ridges in the subsurface. This feature called "Texel High" dominated the coastal evolution in the area for a long time.

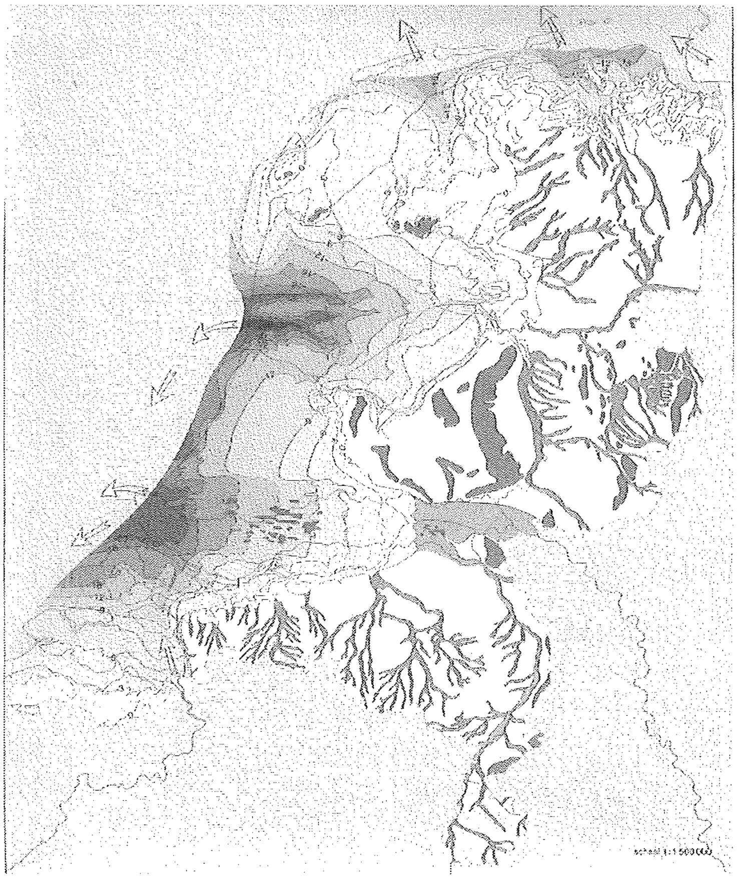

Figure 2-12 The Netherlands during Pre-Boreal: 7500-8500 BC (Zagwijn, 1991)

The fast rising sea level resulted in an overall retreat of the coastline, since the available amounts of sediment were too small to stabilise the coast. The transgression of the sea took place via the valleys in the late-glacial landscape. These valleys changed into lagoons and estuaries that became major sinks for sediments of predominantly marine origin (compare Figure 2-13 and Figure 2-14). In periods of rising sea level, a coastal plain has the tendency to maintain a more-or-less constant level of the shoals with reference to the mean sea level by accumulating sediment. Repeated flooding of the inter-tidal areas deposited sediments on the shoals. Moreover, fine-grained sediments tend to accumulate in lagoons. The sediment originates from the surrounding coasts that show a tendency to erode in the vicinity of the tidal inlets. At present we term this phenomenon the "sand hunger of the Wadden Sea". The sediments were transported towards the tidal basins by wave-driven longshore and cross-shore transport and by tidal action. The rivers Rhine and Meuse deposited sand and clay in their valley, thus keeping the invading sea out.

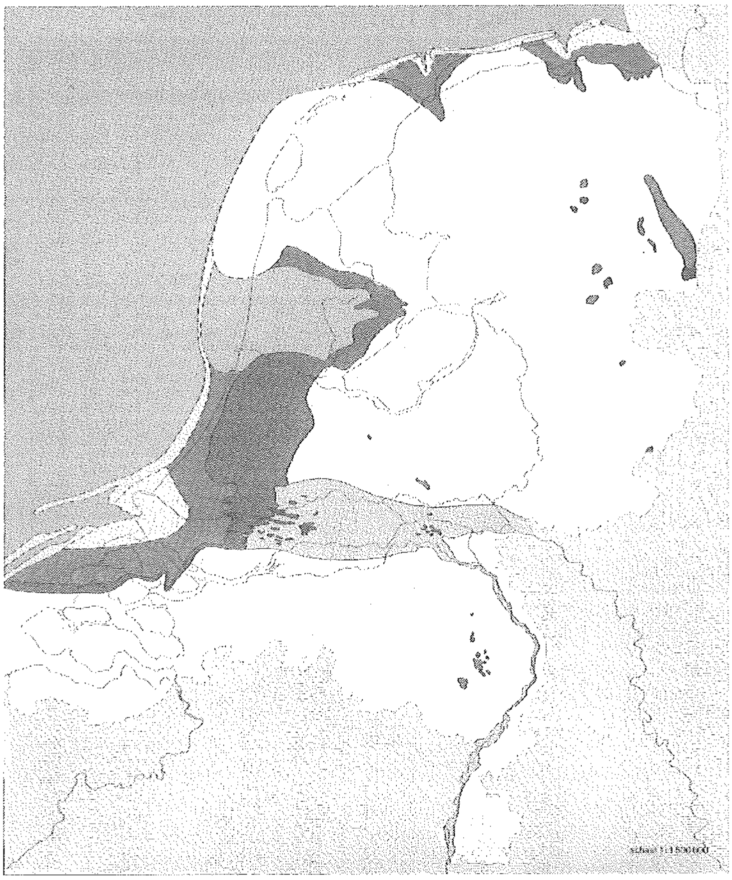

Figure 2-13 The Netherlands during Early Atlanticum: 5500 BC (Zagwijn, 1991)

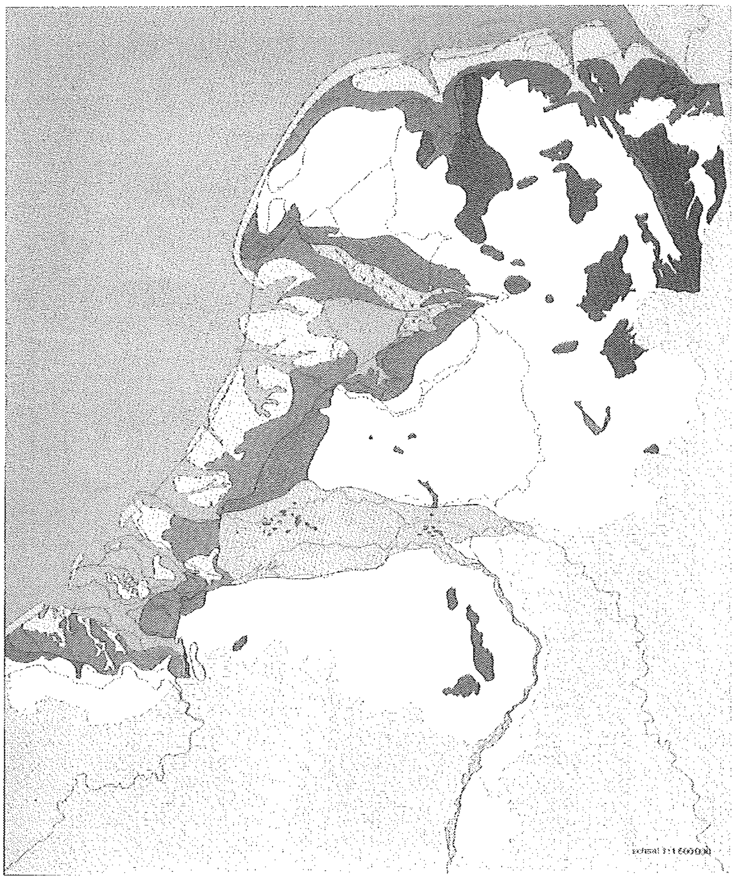

Figure 2-14 The Netherlands during Late Atlanticum: 4100 BC (Zagwijn,1991)

At the end of the Atlantic period, about 6000 to 5500 BP, the rate of sea-level rise had declined significantly (Figure 2-10). This caused a change in the coastal evolution. The tidal basins in Zeeland and South- and North-Holland (up to Alkmaar) started to silt up. The sediment supply gradually became sufficient to fill in these basins, which resulted in closure of the tidal inlets. Subsequently, the coastal barrier started to prograde to the west. Eventually this resulted in a closed coastline that was only dissected by the mouths of the rivers Scheldt, Rhine/Meuse, the Old Rhine near Leiden and the Oer-IJ near Castricum (Figure 2-15).
Behind this coastal barrier the marine influence had completely disappeared and large-scale peat formation started. The coast between Alkmaar and the present island of Vlieland was still dominated by the Texel High. This 'high' caused to coastline to retreat much more slowly than it did to the south of it. Consequently, this part of the coast was a promontory. The sediment that was eroded from the Texel High was transported to the north east and south, into the tidal basins. The tidal basins east of Vlieland never silted up completely. This means that there has always been a predecessor of the present-day Wadden Sea (compare Figure 2-15 to Figure 2-17).

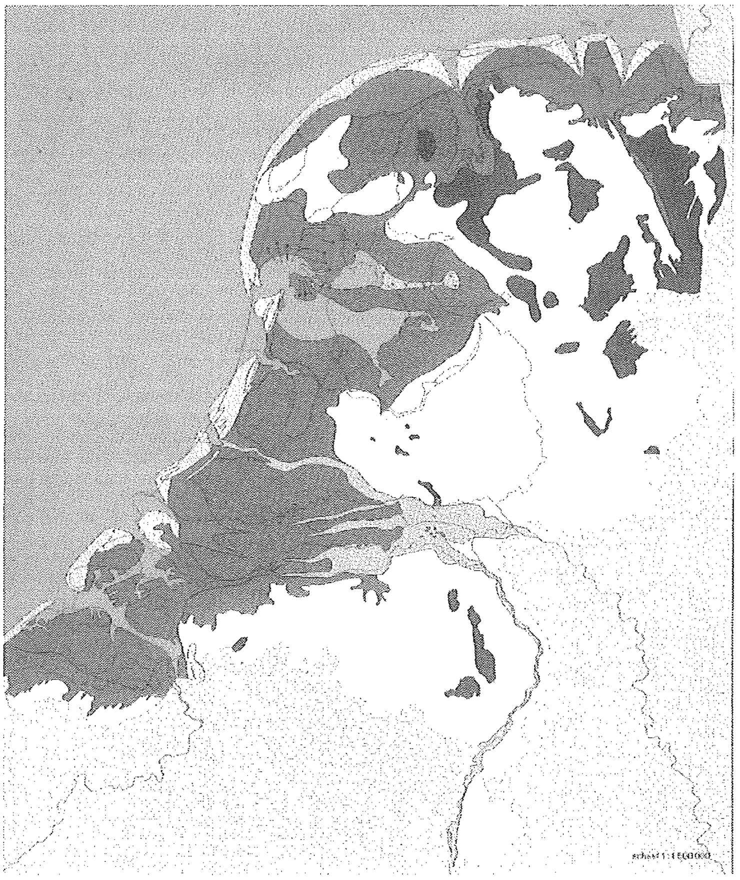

Figure 2-15 The Netherlands during Early Sub-Boreal: 3000 BC (Zagwijn, 1991)

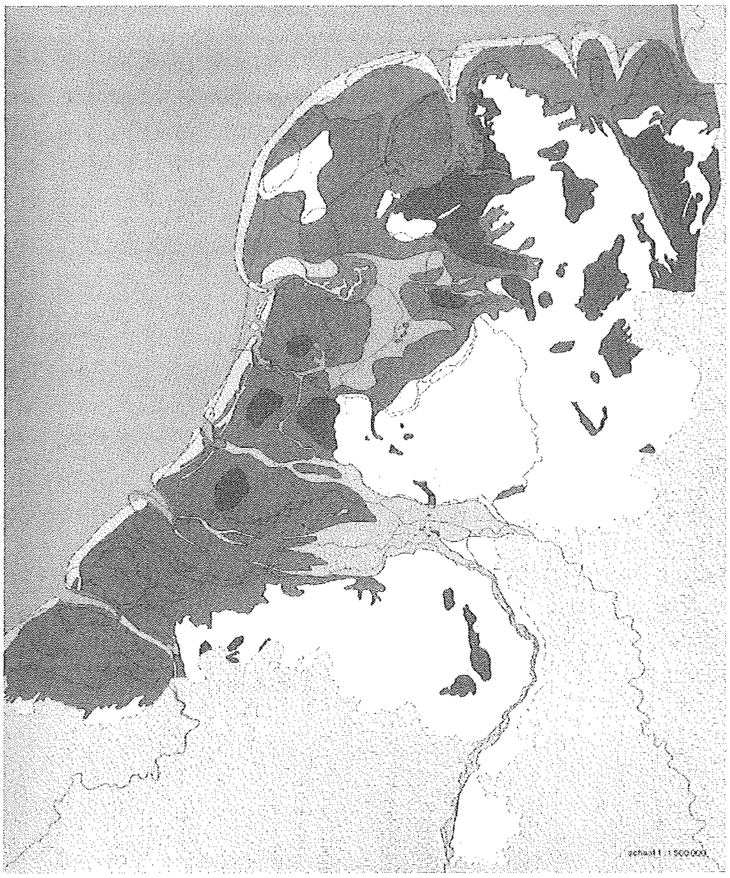

Figure 2-16 The Netherlands during Mid Sub-Boreal: 2100 BC (Zagwijn, 1991)

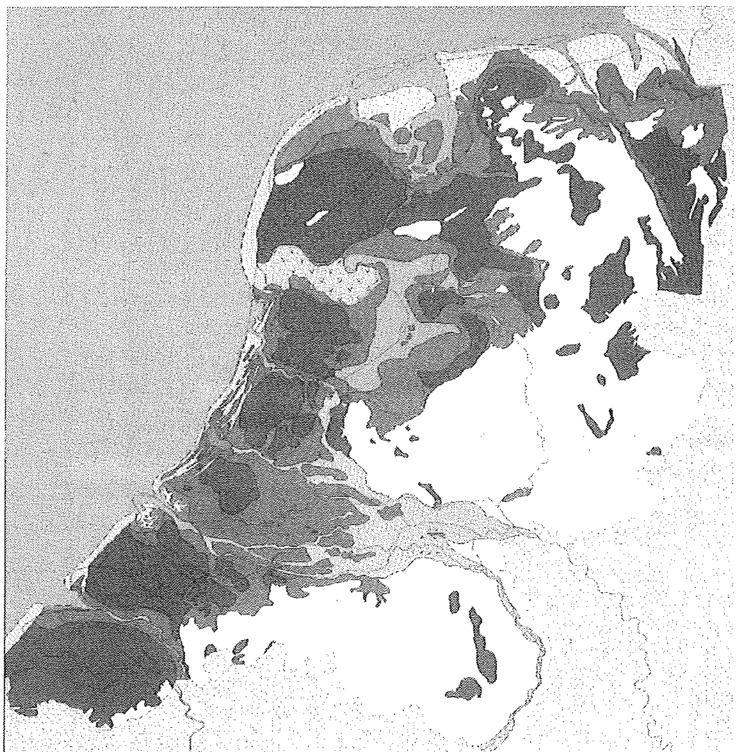

Figure 2-17 The Netherlands during Late Sub-Boreal: 1250 BC (Zagwijn, 1991)

From 3000 years ago onwards developments in our region were no longer determined by the forces of nature alone. The conditions for human settlement improved and gradually the land became inhabited by people who increasingly interfered with the flow of water and the flow of sediment. Thus, developments from about 1000 BC onwards included a mixture of natural and human influences, the human factor becoming more important as technical skills gradually increased. Developments after 3000 BC were no longer revealed only by geology but by archaeology as well, later supported by early written history.

The Subatlanticum, from 2900 years ago up to the present, is characterised by an even slower rate of sea level rise. Although one would expect that this would lead to further consolidation of the coastline, that did not occur. Subsidence of the areas behind the coastal barrier led to new incursions of the sea, resulting in the development of new tidal basins. In Zeeland, this eventually resulted in the formation of large estuaries. The coast of Holland between Schouwen and Alkmaar stopped prograding. The delta of the Old Rhine was eroded after the abandonment of this river course. This stretch of coast started eroding slowly, while the remainder of the Texel High, between Alkmaar and Vlieland was finally flooded, creating the western Wadden Sea. Several breaches developed. One of them expanded at the cost of the others, and finally became the Marsdiep, the largest tidal inlet of the present-day Wadden Sea. The Marsdiep connected the Almere, a large inland lake, with the North Sea. The Vlie formed another connection, to the north. The eastern part of the Wadden Sea expanded again (see Figure 2-18 to Figure 2-20).

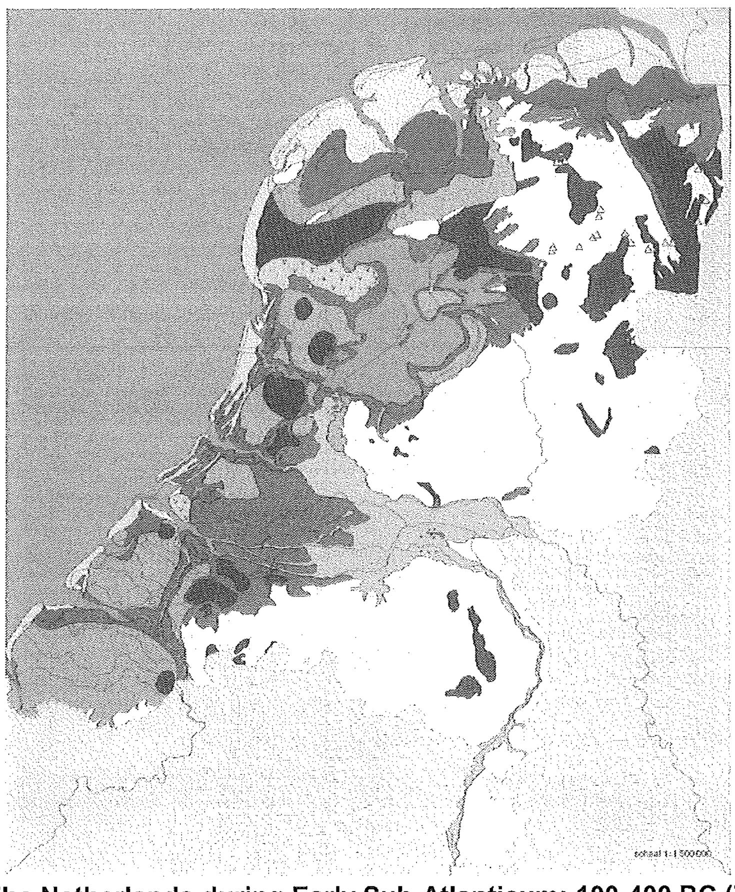

Figure 2-18 The Netherlands during Early Sub-Atlanticum: 100-400 BC (Zagwijn,1991)

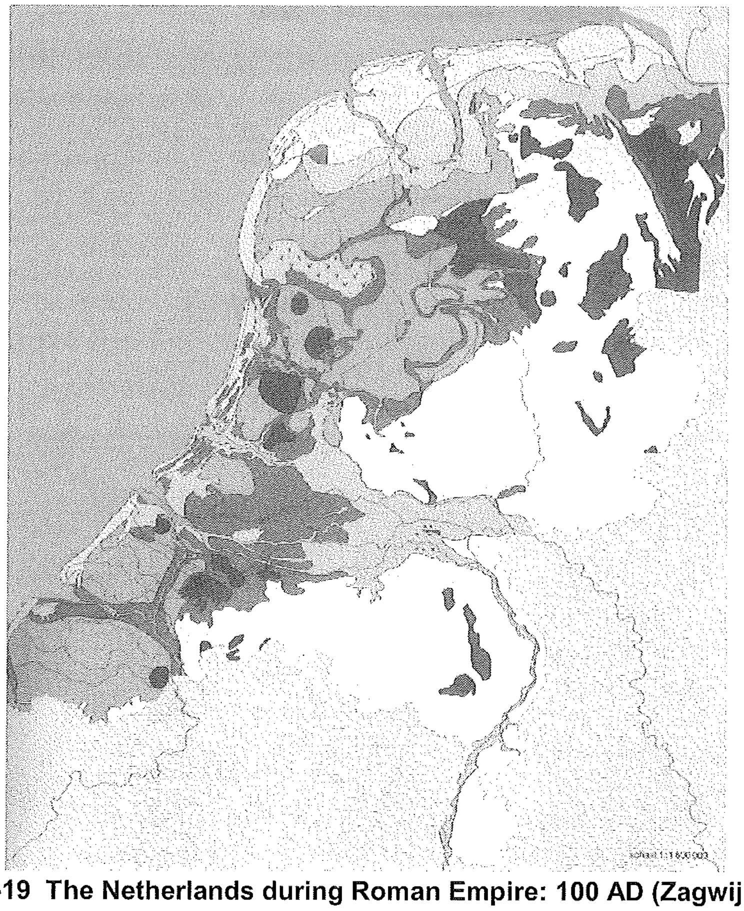

Figure 2-19 The Netherlands during Roman Empire: 100 AD (Zagwijn, 1991)

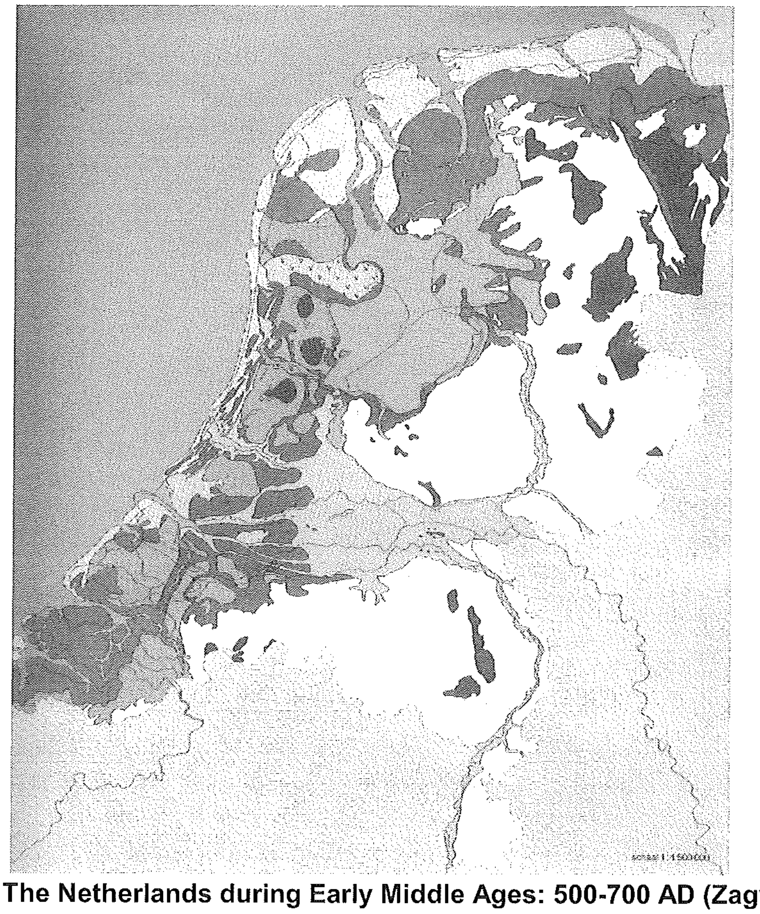

Figure 2-20 The Netherlands during Early Middle Ages: 500-700 AD (Zagwijn, 1991)

As a result of the new inlets, extensive peat areas were being drained, partly by a natural system of creeks and gullies, partly by human interference that becomes noticeable from the Roman age (say 2000 years ago). The better drainage of the peat areas also caused some settlement. Slightly later during this period, the coastal erosion was interrupted in some way, since we know that during the Roman age river mouths and tidal inlets (Rhine near Katwijk and "Oer IJ") were choked. (Even at present we can distinguish the course of the "Oer IJ" in the vicinity of Amsterdam by the deep location of firm foundation layers). A remarkable event in this period also was the creation of the Western part of the Wadden Sea by inundation of the massif around Texel. The sea level in this period must have been around NAP - 0.5m (see Figure 2-19).

2.5.2 Human intervention

The situation described at the end of section 2.5.1 is approximately the situation of the country when the Romans entered and built their fortresses at strategic points along the seashore and the riverbanks. In the same period, inhabitants of other origin started to manipulate the water management in the lagoon area behind the dunes by damming small creeks and by constructing small-scale discharge sluices. Excavations near Vlaardingen show primitive hydraulic engineering works consisting of piled revetments and hollow trees to drain water dating from as early as 100 AD.

Centuries later, medieval rulers constructed dams in larger creeks and rivers thus influencing the distribution of water over the various outlets of the main rivers. The purpose of the dams varied, some were meant to enhance land traffic, others were meant to obstruct navigation in order to levy taxes or to prevent salt water intrusion. The result was not only a change in the hydraulic conditions, but also a change in the sediment distribution, with much of the sediment trapped behind the dams and not reaching the coast.

Thus in two ways (by natural influences and by human interference), the extra sand sources, which had made the growth of the Dutch coast possible, became exhausted. As the sea level continued rising, the demand for sand also continued. The amount of sediment carried by the rivers became insufficient and the coast retreated again. This is the reason why Roman fortresses in front of Katwijk and in various parts of Zeeland are now inundated. The situation at the beginning of the Middle Ages is shown in Figure 2-20. It is remarkable that in spite of the erosion the barrier in the West remains in tact, whereas the gully system in the North erodes further from the Wadden Sea into the later Zuiderzee and Lake IJssel.

During the ninth century AD, extensive cultivation of the peat lands, which had been formed under the favourable climatic conditions, started. The cultivation of the land was necessary because of the population pressure. Large parts of the land consisted of peat masses, situated some (1-3) meters above sea level, and thus not too vulnerable to flooding during storms. In the North and the South, marine sediment fields were found, which were inundated only during high floods. Peat and marine sediment deposits were a buffer against the sea. At that time, the coast was closed, apart from a few river mouths and sea arms. There were sand dune rows, parallel to the coast (Old Dunes).

In the eleventh century, the rate of population growth increased. Peat excavation for heating and salt extraction weakened the water-weir function of the peat lands. Cultivation of the land was further intensified. In the case of peat lands, artificial drainage (first by gravity, later by windmills) led to greater land subsidence; this contributed to the growing vulnerability to inundation and an ever-increasing need for artificial drainage.

Between the twelfth and the fourteenth centuries, the extensive peat area between the Old Dunes and the Utrechtse Heuvelrug was cultivated (Grote Ontginning = Big Cultivation). In the thirteenth and fourteenth centuries, this land subsided so much (sometimes even 4 to 5 meters), that it became situated at or even below sea level. Drainage was a growing problem; therefore agriculture yielded to cattle breeding. The sea level rose, the land subsided, and lakes were formed due to peat mining. Rivers caused inundations, as the upper parts of their catchment areas were also affected by deforestation and subsequent cultivation.

Weakening of the sandy coast by erosion and subsidence of the land behind the barrier coast created the boundary conditions for major flood disasters during storm floods. The dune coast gave way several times in the late Middle Ages. Extensive areas in the lower part of Holland were inundated and lost to the sea. In this way, large parts of the province of Zeeland were lost and the Zuiderzee (now Lake IJssel) was formed, or rather extended (Figure 2-21). In spite of the ongoing erosion of the sandy coast in the Middle Ages, new dunes were formed all the time in quieter periods. The mechanism of these slightly contradictory developments is still not completely clear.

The threat of the sea did not discourage the population. On the contrary, during the first half of the thirteenth century those threatened by the frequent floods formed the oldest more or less democratic institutions in the country: the Water Boards (Dutch: Waterschappen). These institutions co-ordinated the efforts to construct dikes and dams. The first polders were created: there were areas enclosed by dikes where a lower artificial water level was maintained than that in the surrounding areas. Water Boards got their own political power, based on the land they "controlled". In this way, the basis was laid for the later Dutch proficiency in dredging and shore protection. Mankind was on the loosing side, however, until the industrial revolution provided powerful tools.

In the first place these tools were powered by steam engines that could replace the windmills. The greater capacity of the steam driven pumping stations facilitated the reclamation of large inland lakes that had become a threat in their own right. Lake Haarlem (Haarlemmermeer) was reclaimed, and so were the creeks between Amsterdam and IJmuiden, along with the construction of the North Sea Canal, giving Amsterdam a modern connection with the North Sea. To stabilise the eroding dune coast, groynes were constructed. The "Hondsbossche Zeewering", a heavy seawall was constructed south of Den Helder.

Figure 2-21 The Netherlands during Late Middle Ages: 1000-1200 AD (Zagwijn, 1991)

Still, the North Sea formed a serious threat. In 1916, large-scale inundations took place in the province of Noord Holland and around the former Zuiderzee. Eventually, these inundations led to the decision to close the Zuiderzee and to construct a number of large polders in the newly formed Lake IJssel. The construction of the closure dam (Afsluitdijk) in 1932 must be considered as the starting point of modern coastal engineering in the Netherlands. This is not only due to the scale of the works, but even more to the application of scientific methods to predict the consequences of such works (see Appendix 2).

In this respect, mention must be made of the work of the Lorentz Commission, which developed a technique for tidal computations to predict current velocities during the closing operation, and also changes in the tides in the Wadden Sea after completion of the closure. Similarly, hydraulic model studies were initiated to study the discharge capacity of discharge sluices in the Afsluitdijk and scour due to tidal currents. These model investigations were conducted at Delft University of Technology under the direction of Prof. Thijsse. A few years later, the laboratory was privatised and it became the well-known Waterloopkundig Laboratorium (WL | Delft Hydraulics).

A new disaster occurred in 1944, when the island of Walcheren was inundated, following bombing of the dikes by the RAF in an attempt to gain access to the Port of Antwerp. The dikes could only be repaired with great difficulty, using techniques that had never been used before, i.e. sinking of large concrete caissons in the gaps of a closure.

Even more serious was the storm flood of February 1953, which caused extensive flooding in the SW part of the country and took over 1800 lives. After repair of the damage, it was concluded that the common practice of raising the level of dikes a little higher than the highest recorded flood level posed great risks. For the first time, a real statistical risk analysis was carried out to establish an acceptably small chance of a new major flood disaster. The Delta commission advised taking a flood level with a probability of exceedance of 10^{-4} per annum as the base for design of the sea defence system. This would require such major strengthening works of the existing dikes in the Delta region that it was considered better to close the estuaries in the SW part of the country. The Delta project was born. It took more than 25 years to finalise the project. A more detailed description is given in Appendix 3.

Coastal erosion continues and therefore the Dutch Government recently decided to maintain the North Sea Coast at its present position by artificial means, these mainly consisting of large beach nourishment projects. The technical background of these works will be explained elsewhere in this book.

2.5.3 Sediment balance

After all these qualitative considerations, it is good to realise how much sediment was moved during the Holocene. We have seen that the Dutch coast is a sediment importing system. During the Holocene 200 to 250 billion m^3 of sediment, consisting of sand (70%), silt (25%) and peat (5%), was deposited. The greater part of the sand was eroded from the Pleistocene area of the present North Sea; about 10% was transported by the Holocene Rhine.

The Holocene is thus characterised by the great mobility of sediments and by a large influx of sediment. The geological division within the Holocene was based mainly on the development of vegetation in North-west Europe after the melting of the ice caps. The different geological periods can be traced in the soil structure. The Holocene sediment covers the area shown in Figure 2-22.

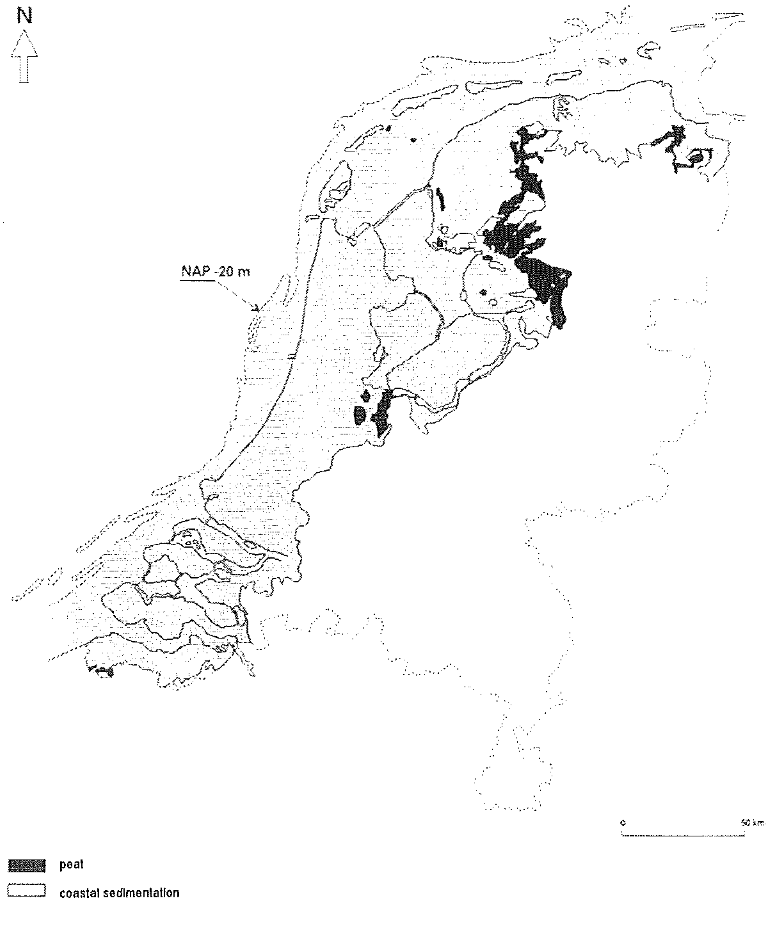

Figure 2-22 Holocene sediments in the Netherlands

The Pleistocene deposits can be found under the Holocene layers in the Western part of the country. On many occasions, these Pleistocene layers form the base for pile foundations, since thick layers of ice have surcharged them. The Pleistocene deposits can still be found at the surface in the eastern part of the country.

The sediment import during the Holocene decreased consistently from an average of ±42 million m^3 per year between 8000 and 5000 BP, via ±27 million m^3 per year between 5000 and 3000 BP to an average of ±7 million m^3 per year after 3000 BP. This seems to be partly linked to the velocity of the rise in sea level. This mechanism is thought to have worked owing to the presence of big sand deposits during fast sea level rise, coupled with the retreating coastline. At a lower rate in sea level rise, the coast could become more-or-less stabilised; remainders of the deposits were exhausted. After that, new sediment could be drawn by new retreat of the coast.

From the geological analysis the conclusion can be drawn that for several millennia the import of sand from outside the coastal system has probably been too little to stabilise the coast. Today the it cannot be anticipated that this import will increase. Recently, a decision was made to maintain the coastline in its present position. This decision implies the necessity to find artificial ways to ensure sand import. At this stage, (1999), about 6 million m^3 per annum is supplied artificially. It must be expected that an acceleration of the rise in sea level under the influence of the greenhouse effect will cause an increasing demand for sand from the Wadden Sea, resulting in stronger erosion of the existing coast. It is expected that improved technical capabilities (dredging) will enable us to cope with this increased erosion. From the geological analysis at the same time it can be learned that major natural changes can take place in a relatively short period with immense effects on the population of the coastal zone.

3. CLIMATOLOGY

3.1 Introduction

You do not need to be a mountain climber to know the effect of the topography on the weather. The presence of mountains, oceans, and other natural features influences the climate of an area, and the climatic conditions influence the topography. In other words: the climate and the topography of a region are closely related to each other.

The climate is important for coastal engineering, as it determines the way in which the naturally available water behaves. This influences the movement of sediments, which has a major influence on the physical properties of the Coastal Zone and on the design of coastal structures.

3.2 Meteorological system

The climate is the sum of the annual effects of the weather. In some area (equatorial rain forests) there is little difference between the data relating to climate and weather. Where there is greater seasonal or daily variation, weather effects may vary greatly.
Therefore, weather effects are quantified by so-called meteorological variables, which are:

1. Temperature
2. Atmospheric pressure
3. Atmospheric humidity
4. Air density
5. Vertical air velocity
6. Horizontal air velocity (wind)

The motor of all meteorological processes on the earth is the energy coming from the sun. The atmosphere and the surface of the earth receive this energy by radiation and lose it in the same way. The energy transformation processes (conversions) in between give a numerical value to the meteorological variables. If the different conversion processes are quantified, an energy balance of the atmosphere can be constructed. This balance shows the different components of the energy cycle, which is governed by the following meteorological equations:

1. the gas law
2. the first law of thermodynamics (heat equation)
3. the equation of continuity (mass conservation)
4. the moisture equation (conservation of moisture)
5. the vertical equation of motion (Newton's second law)
6. the horizontal equation of motion (Newton's second law)

Given the six variables and the six equations, in principle it is possible to solve meteorological problems by integrating the equations from a given state forward. In this integration, proper boundary conditions must be applied at the bottom and top. Finally, when the domain of interest does not extend around the globe, lateral boundary conditions have to be prescribed as well.

3.3 From meteorology to climatology

In order to quantify a climate, it is usual to average the weather effects over 30 years. In addition to the average values of the meteorological variables, other values are needed in order to characterise a climate properly, especially for engineering purposes. For example monthly or annual minima, maxima, and threshold values for a given lifetime are necessary statistical information.

Primary sources for climatological data are the monthly tables in the archives of meteorological services. Others are bulletins and yearbooks for meteorology. Climate atlases and global climate maps are also available.

Going from meteorology to climatology, we see the time scale growing (via statistics). A somewhat comparable step can be taken with respect to the spatial dimensions. It is also possible to make generalisations in the case of many spatial processes. Many of these are described in literature. In this section, a few processes only are summarised briefly (Harvey [1976]):

1 the hydrological cycle and cloud formations
2 solar radiation and temperature distributions
3 pressure gradients and winds
4 atmospheric circulation

3.4 The hydrological cycle

The cyclic stages and processes of water are drawn in Figure 3-1.

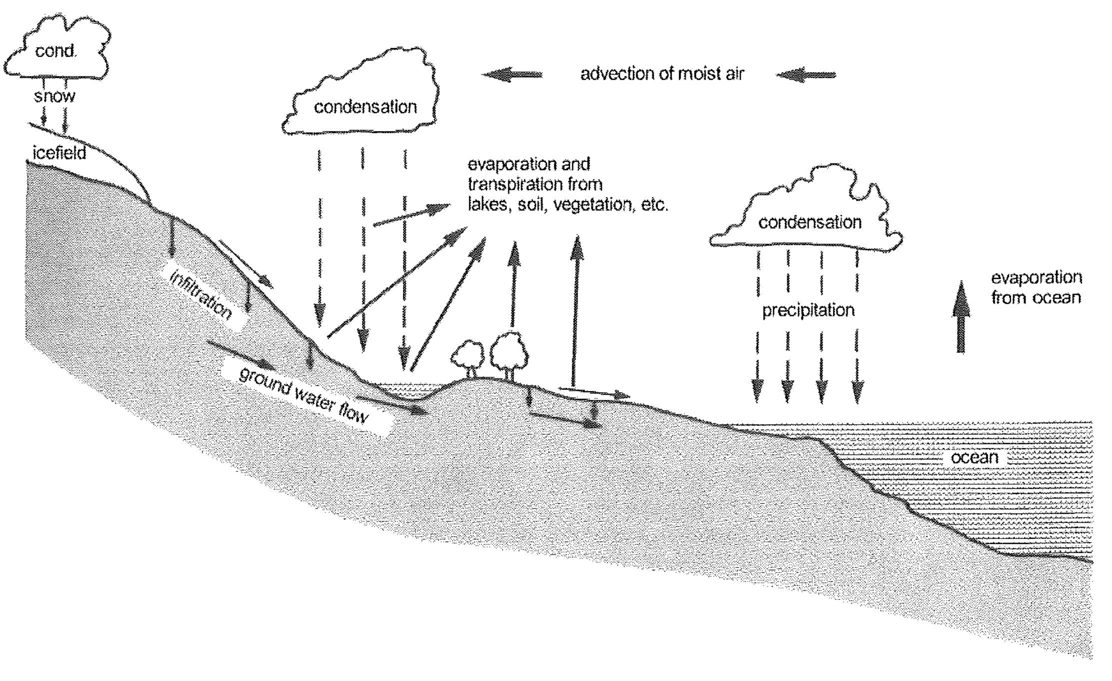

Figure 3-1 The hydrological cycle

The process whereby water is transferred from ocean and land surfaces into the atmosphere is known as evaporation. When it occurs from plant surfaces it is called transpiration, and when it occurs directly from an ice or snow surface to the vaporous state it is known as sublimation. The water vapour, which is thus added to the gases in the atmosphere, increases the pressure within the atmosphere. The part of the total pressure that is attributable to the water vapour is referred to as the vapour pressure (e). An alternative way of specifying the amount of water vapour present in the air is by using the humidity mixing ratio, which is the ratio of the mass of water vapour and the mass of dry air.

The opposite process to evaporation is condensation. When the processes of evaporation and condensation balance one another, equilibrium is reached; the air is said to be saturated with water vapour. The pressure at which this is the case is called saturation vapour pressure e_w. This saturation vapour pressure is very temperature-dependent, increasing more and more rapidly as temperature increases. Therefore, when, while cooling an amount of partly saturated air, the dew point is reached, that is the temperature at which the air is fully saturated (at constant pressure). When there is no surface of any kind for water to condense on, air can become supersaturated and still retain its water vapour. A measure of the amount of water vapour in the air is the relative humidity (U).

$$U = \frac{e}{e_w} \cdot 100\% \qquad (3.1)$$

where:
U = relative humidity (%)
e = water vapour pressure (mb)
e_w = saturation vapour pressure (mb)

The relative humidity is increased not only by an increase in the water vapour content, but also by a decrease in temperature (if the water vapour remains constant). For this reason the diurnal variation in relative humidity often mirrors the diurnal variation in air temperature.

Although no surfaces seem to be available in a free cloudless atmosphere, there are many impurities such as salt particles from the evaporation of sea spray, dust from deserts and volcanic eruptions and smoke from fires on which condensation can take place. These are known as condensation nuclei. On most types of nuclei, condensation already takes place below a relative humidity of 100%.

The saturation of air leading to condensation is usually caused by air being cooled. This cooling happens, for instance, when air rises, or due to the daily temperature fluctuation. There is another important process leading to condensation, which is illustrated by Figure 3-2.

Consider samples of air represented by points D and E. Neither is saturated with water vapour, but if they are thoroughly mixed together in equal quantities the resultant mixture will be represented by point F - which is saturated. Hence it is seen that mixing of two different types of air can lead to saturation and condensation.

Back to cooling of the air by air ascent. There are three principal causes of air rising in the atmosphere:

1. When air which is moving horizontally, encounters a hill or mountain range, it must rise to continue
2. Horizontal convergence of air, which can lead to uplift of the warmest (lightest) air (frontal uplift)
3. Convection by warming of the air near the ground (making it less dense)

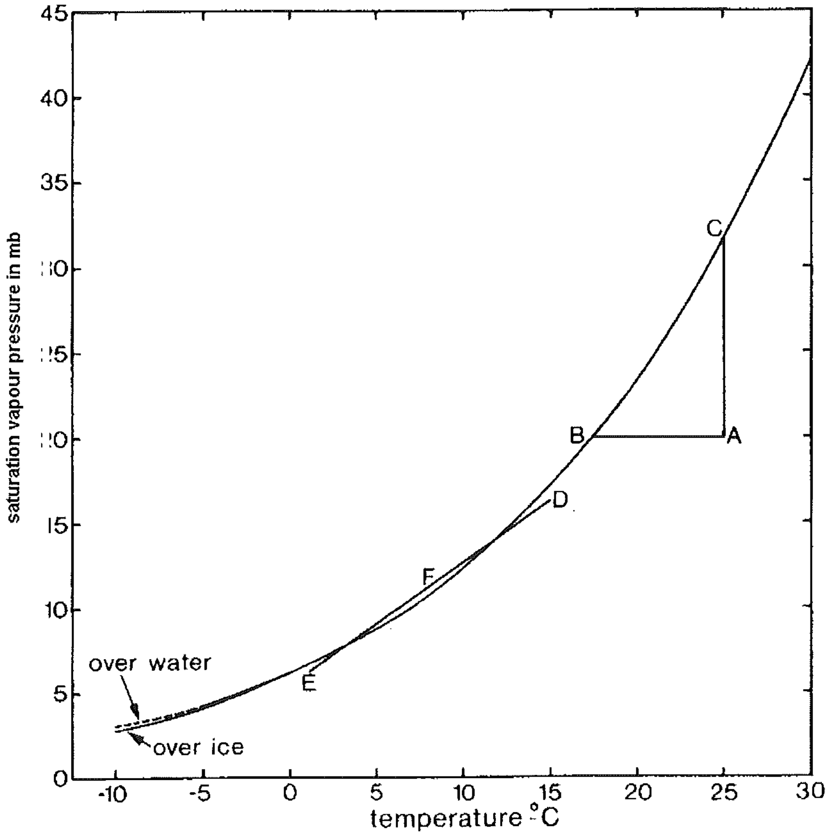

Figure 3-2 Saturation vapour pressure as a function of temperature

3.5 Solar radiation and temperature distribution

The sun emits electro-magnetic radiation, which is the main source of thermal energy for the earth. The intensity of the radiation coming from the sun is denoted by E and expressed in energy per unit surface area. It can be calculated by using Stefan's Law:

$$E = \sigma T_s^4 \tag{3.2}$$

where:

σ = constant of Stefan-Bolzmann = 5.67 * 10^{-8} W m^{-2} K^{-4}

T_s = absolute temperature of the sun surface, which can be considered to be 6000 K.

Using Equation (3.2), the amount of radiation per unit surface area is 3.402 * 10^{-2} W/m^2. This sun radiation is divided over a range of frequencies or wavelengths (Figure 3-3).

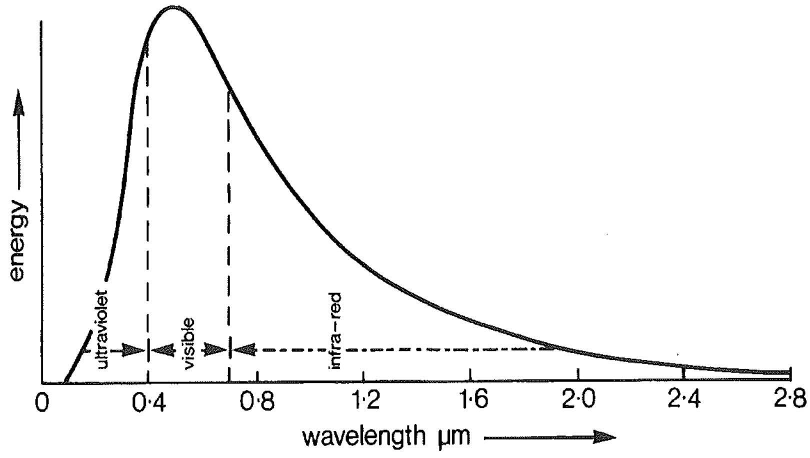

Figure 3-3 Distribution of radiation intensity over wavelength for a black body with a surface temperature of 6000K (sun)

The radiation that reaches the earth, including its atmosphere, depends on the varying distance between sun and earth. As the radiation then passes through the atmosphere, it is subject to absorption, scattering, and reflection by clouds (Figure 3-4). The proportion that the cloud (or another surface) reflects is called its albedo.

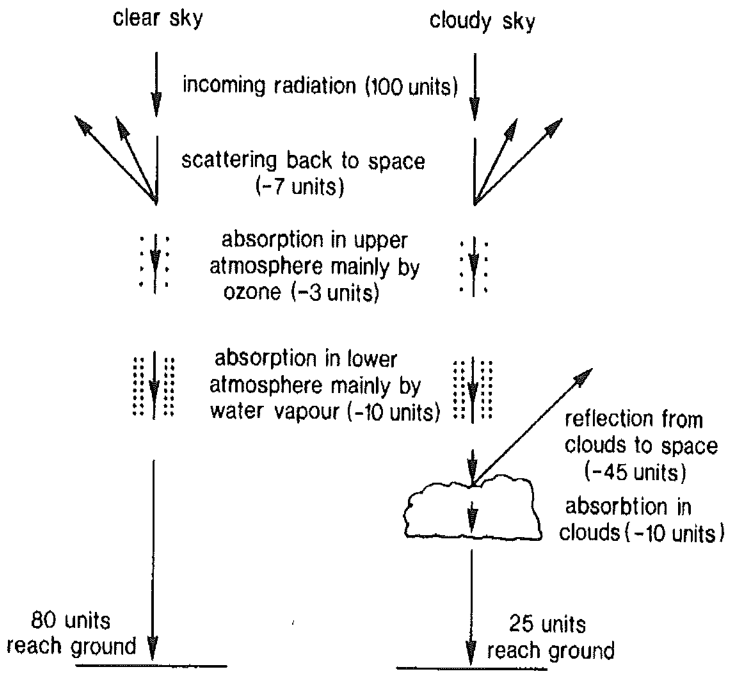

Figure 3-4 Reduction of solar radiation intensity as it is transmitted through the atmosphere

The radiation which reaches the earth surface may be absorbed there, be transmitted downwards if it encounters a material which is transparent to it, or be reflected. The albedo of the surface depends on its substance and texture, on the angle of incidence of the radiation, and on the wavelength of the radiation. The absorption of radiation leads to heating. The heat may be transmitted downwards by conduction or, in the case of fluids, by convection.

If the earth continued to absorb solar radiation without any loss of heat, its temperature would rise indefinitely. This does not happen, because the earth, in its turn also emits electro-magnetic radiation into space. Taking mean annual values, and ignoring any change in the earth's mean annual temperature from one year to the next, a balance must exist between incoming solar radiation and outgoing terrestrial radiation.

The earth mainly emits visible and infrared radiation (wave lengths > 4 μm). The gases in the atmosphere, which absorb this low-frequent terrestrial radiation, are water vapour, carbon dioxide and ozone. They emit long wave radiation in all directions, which is called secondary reflection. They therefore act as a layer of insulation around the earth analogous to the glass of a greenhouse, and their effect on earth temperatures has been called the greenhouse effect.

The earth follows an elliptical path around the sun; its mean distance being about 150 million km, but this varies at the present time by about 5 million km in the course of a year. The amount of radiation received in a day depends upon the length of time the area is exposed to the sun's rays, the angle between the sun's rays and the earth's surface, and the distance of the earth from the sun. These factors vary with latitude and season.

The process of absorption and reflection leads to distinct differences at distinct locations around the globe. At high latitudes (near the poles), the incoming radiation is less than the outgoing radiation: a net loss of heat by radiation is found. At low latitudes (near the equator), there is a net gain (Figure 3-5). Horizontal transfer (advection) of heat is the result. The change-over from a surplus to a deficit in the net annual radiation balance occurs at about 37^0 latitude N and S. The winds and ocean currents are responsible for the advective heat transport. These heat transport processes themselves are generated by the uneven distribution of heat over the earth's surface.

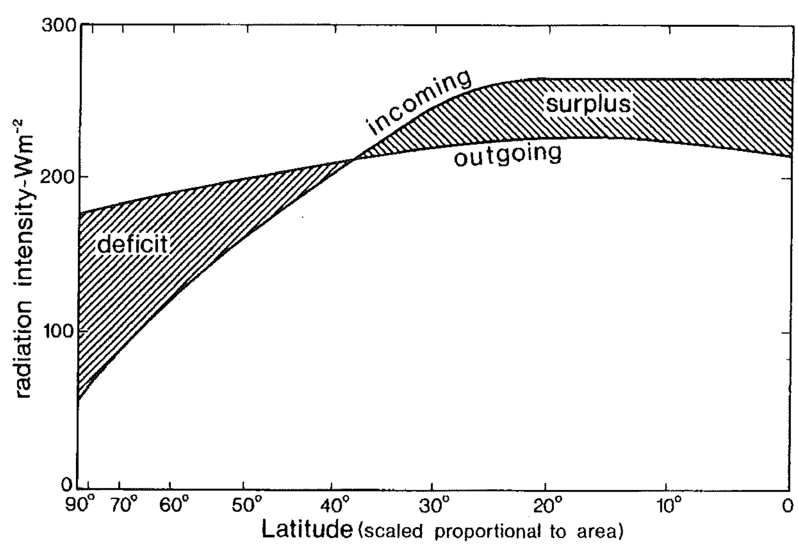

Figure 3-5 Long-term average incoming and outgoing radiation intensity

In short, the days and nights, and the seasons, cause variations in temperature. An ocean responds differently to these variations than a continent does. In water, the solar radiation penetrates further than in land; water has a greater heat capacity than land; water has a big storage possibility for heat by the process of mixing and evaporation. These differences between water and land cause differences in the air temperature distribution over the earth surface (Figure 3-6).

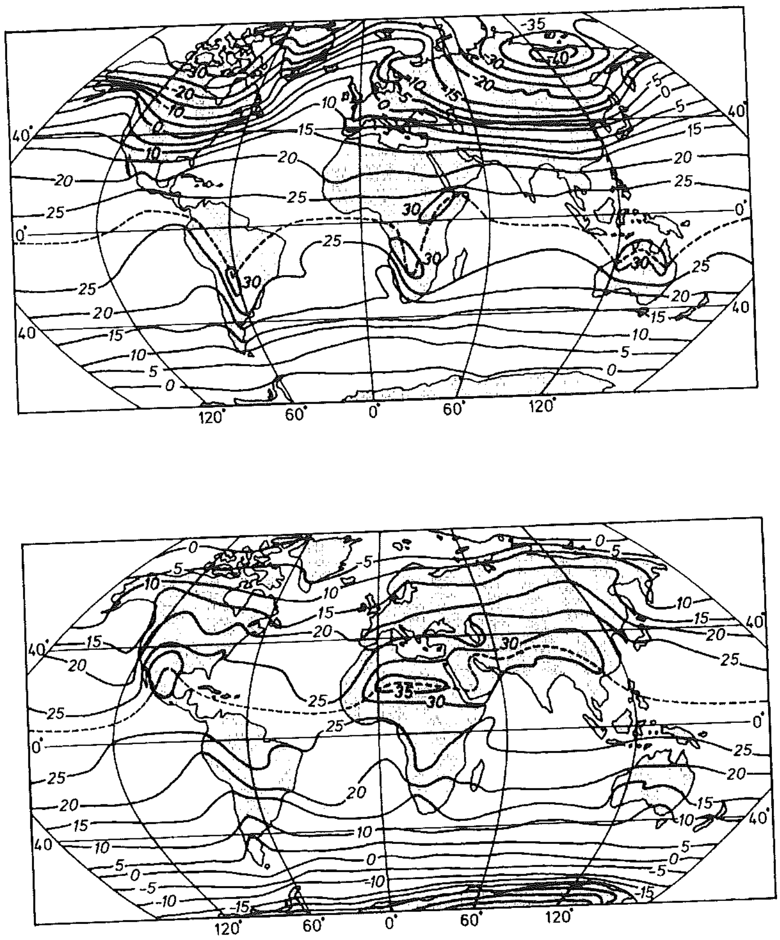

Figure 3-6 Air temperatures reduced to sea level in January and July

The distribution of air temperature over the earth's surface depends on four major factors:

1 Latitude
2 Altitude
3 Nature of the surface, in particular the distribution of land and sea
4 Advection of heat by winds and currents

The advective transport by winds will be discussed in this Chapter; the Ocean Currents will be treated in Chapter 4.

3.6 Atmospheric circulation and wind

If the earth did not rotate, and if its surface albedo would be entirely uniform, transparent to solar radiation, heat capacity and thermal conductivity, then we might expect a simple convection cell circulation to exist within the troposphere[2] in each hemisphere (Figure 3-7). Each cell would have a horizontal dimension of the order of 10^4 km compared with a vertical dimension of only some 10 km. In practice, the cells on the N and S hemisphere are breaking up in three smaller cells each (Figure 3-8).

2 Troposphere is that part of the atmosphere where the temperature decreases with increasing altitude

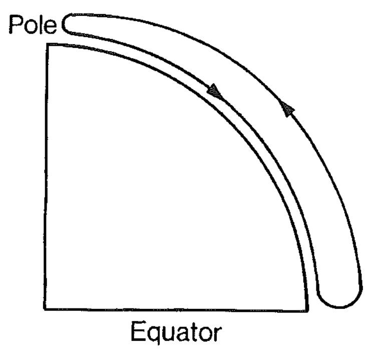

Figure 3-7 Convection cell circulation on a non-rotating uniform earth

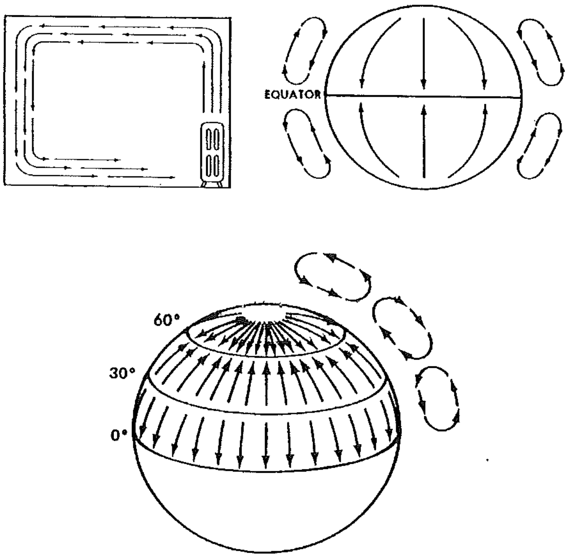

Figure 3-8 Simple three-cell convection

Without earth rotation, a symmetrical global atmospheric circulation pattern could be expected. However, this symmetry is disturbed by rotation of the earth. The *Coriolis* effect, which is the result of the earth rotation, works in different directions in each hemisphere. It causes a deviation to the right (starboard side) on the N. hemisphere and a deviation to the left (port side) on the S. hemisphere. This is also referred to as the Law of Buys Ballot. Therefore, the global atmospheric circulation system is asymmetrical (Figure 3-9). It consists of three major cells on each hemisphere. The lowest latitude cells are called Hadley Cells. This series of pressure belts and wind systems is kept going by this so-called **A-engine** (a combination of solar radiation and earth rotation). In Figure 3-9, one can clearly distinguish regions with mainly westerly winds at latitudes between 30^0 and 60^0 , which we know extremely well in the Netherlands. Also the region with mainly NE and SE trade winds near the equator are obvious.

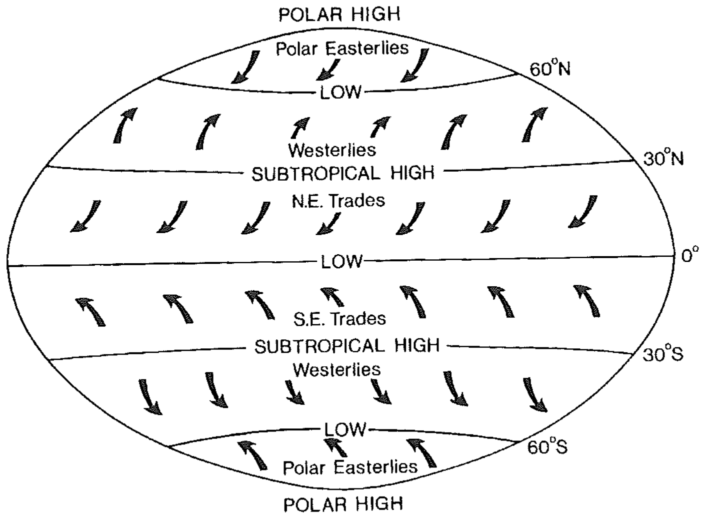

Figure 3-9 Schematic presentation of pressure belts and wind systems at the earth surface

When the non-uniformity of the earth's surface is introduced, the situation becomes considerably more complex, and the influence of the continents can be found. The influence of the ocean/continent contrasts is called the **B-engine**. The seasons can generate thermal effects; for example, pressure systems can be stable during the summer and alternate into another relatively stable configuration during the winter. This is very evident in SE Asia, where the Asian continent warms up in July, thus creating a Low above China, causing a SW wind. In January, when the water of the Indian Ocean maintain a higher temperature than the continent, the situation is reverse, causing a NE wind. Seasonally reversing winds correlated with this seasonal change are called monsoons.

The last major influence on the climate is the topography of a certain area. Mountains affect the pressure distribution in their own way. Local phenomena like land or sea breezes together form the **C-engine**. The most terrifying phenomenon coming from this is the hurricane, which develops above the ocean. It follows a path, which is partly predictable, and is stopped only after crossing into a continent.

The combination of the A-engine and the B-engine leads to very typical wind patterns for the months of January and July (Figure 3-10). It can be clearly observed that some tropical areas are dominated by trade winds (blowing the same direction throughout the year), whereas other areas are dominated by reversing winds (monsoon). It will be clear that such observations will have a strong influence on civil engineering aspects.

Although wind conditions cannot be predicted accurately long in advance, wind conditions can be described statistically. The wind climate consists of both velocity data and directional data. Velocities can be expressed by wind speed (when measured) or as a certain number on the Beaufort scale (when visually observed). These data can be found in meteorological yearbooks and in various atlases. As to the latter, reference is made to specific hydrographic atlases that contain data collected at sea.

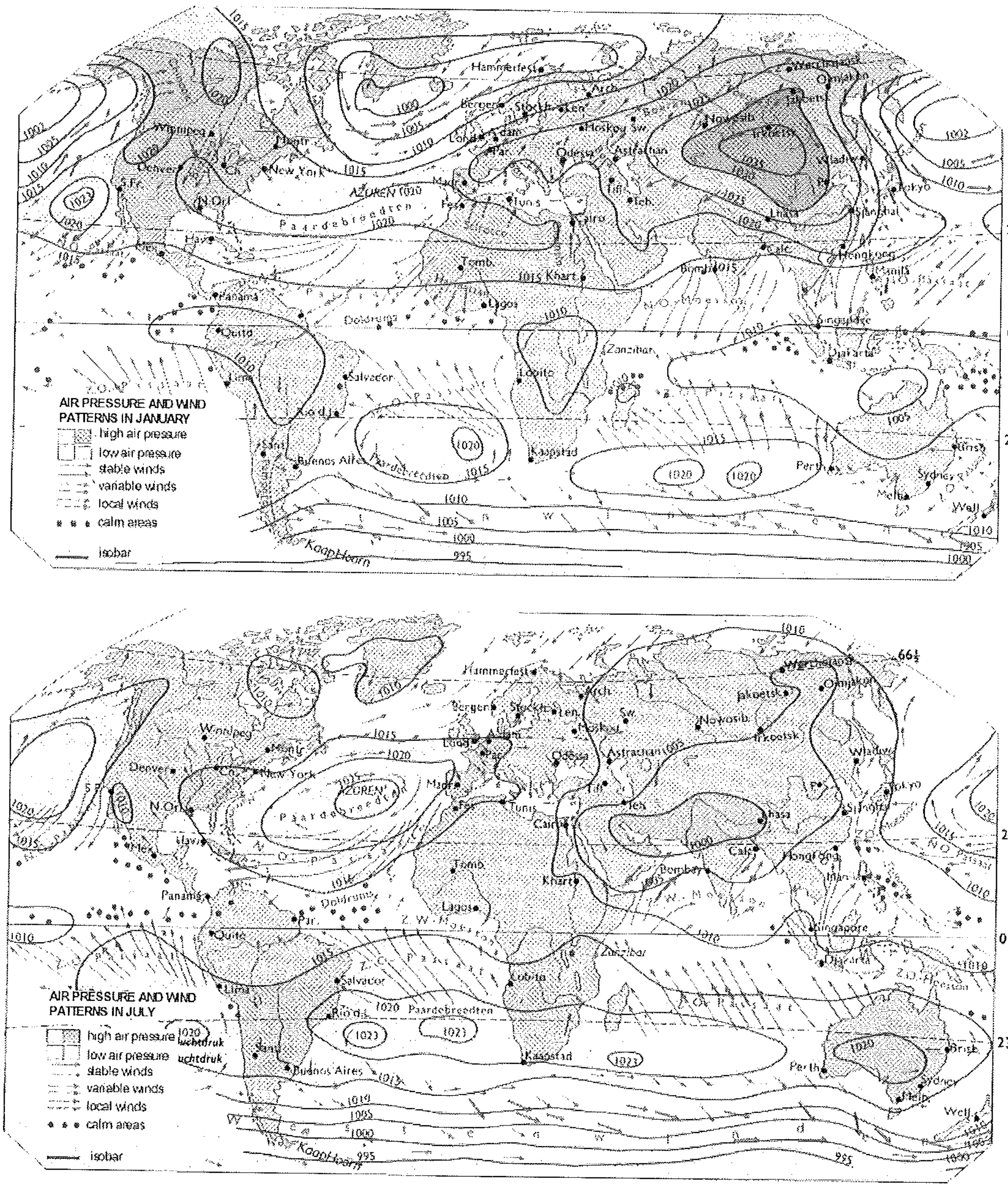

Figure 3-10 Global wind patterns in January and July

Beaufort, a British naval officer, introduced the Beaufort wind scale in 1805. It is still used at present. For tactical reasons the scale was intended to exchange objective information between sailing vessels of the British Navy. The lower scales (2 to 4) refer to sailing speeds of the common naval vessel of that time (man-of-war) under full sail. The intermediate scales (5 to 9) refer to conditions that required reefing of sail. The higher scales (10 to 12) deal with survival of ship and crew. The crews of modern pleasure craft may find the definitions of Beaufort a little rough as is indicated in Figure 3-11. The Beaufort scale is summarised in Table 3-1 in the form that is used at present. Bold printed expressions refer to the official terms of the World Meteorological Organisation (WMO). Standard pictures are available to illustrate the descriptions of the sea surface that have been added to assist observers on board of sea-going vessels to report accurately.

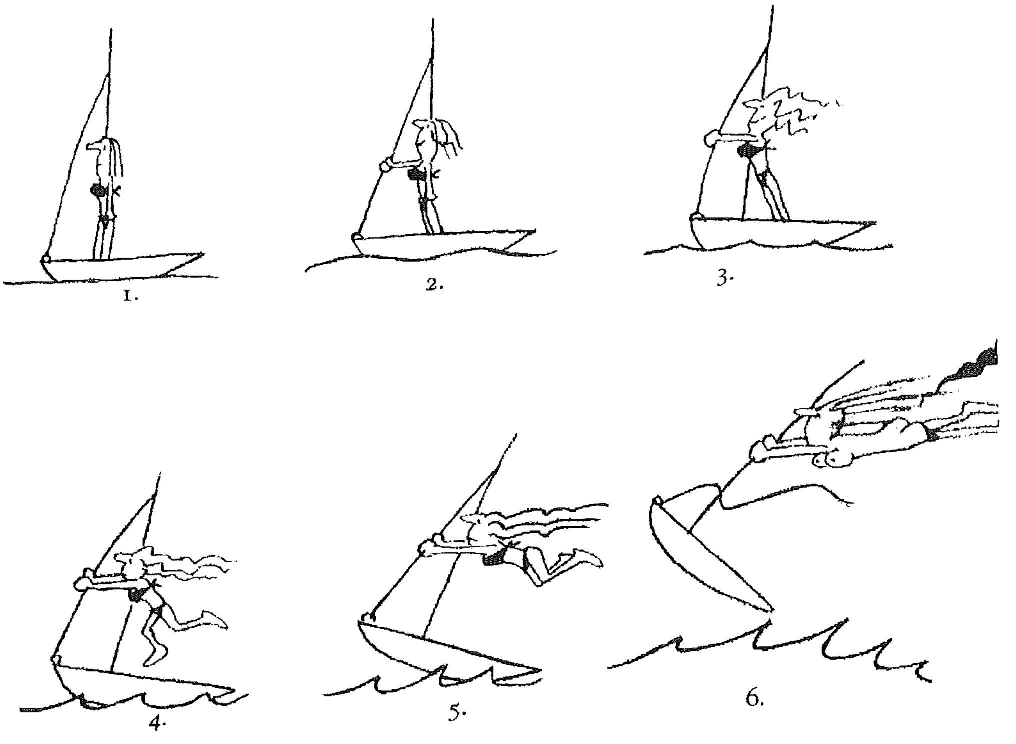

Figure 3-11 Beaufort scale for modern pleasure craft and their crews (after Beard and McKie, 1981)

<table>
<tr><th rowspan="2">Beau-fort No.</th><th colspan="2">Wind Speed</th><th colspan="5">Description</th><th>Wave Height</th></tr>
<tr><th>m/s</th><th>Knots</th><th>Phenomena observed on land</th><th>State of the sea surface</th><th colspan="2">As used by Navy in time of sailing vessels</th><th>In Dutch as used by KNMI</th><th>m</th></tr>
<tr><td>0</td><td>0 – 0.2</td><td>0 – 1</td><td>Calm: Still; Smoke will rise vertically</td><td>Sea like mirror.</td><td colspan="2"></td><td>Windstil</td><td>0</td></tr>
<tr><td>1</td><td>0.3 – 1.5</td><td>1 – 3</td><td>Light Air: Rising smoke drifts; weather vane is inactive</td><td>Ripples with appearance of scales; no foam crests.</td><td colspan="2">Just sufficient to give steerage way.</td><td>Zwakke wind</td><td>0.1 – 0.2</td></tr>
<tr><td>2</td><td>1.6 – 3.3</td><td>3 – 6.5</td><td>Light Breeze: Leaves rustle, can feel wind on your face, weather vane is inactive.</td><td>Small wavelets; crests of glassy appearance, not breaking.</td><td rowspan="3">A man-of-war with all sail set and clean full would go in smooth water from:</td><td>1to 2 knots</td><td>Zwakke wind</td><td>0.3 – 0.5</td></tr>
<tr><td>3</td><td>3.4 – 5.4</td><td>6.5 – 11</td><td>Gentle Breeze: Leaves and twigs move around. Light weight flags extend.</td><td>Large wavelets; crests begin to break; scattered whitecaps.</td><td>3 to 4 knots</td><td>Zwak tot matige wind</td><td>0.6 – 1.0</td></tr>
<tr><td>4</td><td>5.5 – 7.9</td><td>11 – 16</td><td>Moderate Breeze: Moves thin branches, raises dust and paper.</td><td>Small waves, becoming longer; numerous whitecaps.</td><td>5 to 6 knots</td><td>Matige wind</td><td>1.5</td></tr>
<tr><td>5</td><td>8 – 10.7</td><td>16 – 21</td><td>Fresh Breeze: Moves trees sway.</td><td>Moderate waves, taking longer form; many whitecaps; some spray.</td><td rowspan="5">A well-conditioned man-of-war could just carry in chase, full and by:</td><td>Royals, etc.</td><td>Vrij krachtige wind</td><td>2.0</td></tr>
<tr><td>6</td><td>10.8 – 13.8</td><td>21 – 28</td><td>Strong Breeze: Large tree branches move, open wires (such as telegraph wires) begin to whistle, umbrellas are difficult to keep under control.</td><td>Larger waves forming; whitecaps everywhere; more spray.</td><td>Single-reefed topsails and top-gal. Sail</td><td>Krachtige wind</td><td>3.5</td></tr>
<tr><td>7</td><td>13.9 – 17.1</td><td>28 – 34</td><td>Near Gale: Large trees begin to sway, noticeably difficult to walk.</td><td>Sea heaps up; white foam from breaking waves begins to be blown in streaks.</td><td>Double reefed topsails, jib, etc.</td><td>Harde wind</td><td>5.0</td></tr>
<tr><td>8</td><td>17.2 – 20.7</td><td>34 – 42</td><td>Gale: Twigs and small branches are broken from trees, walking into the wind is very difficult.</td><td>Moderately high waves of greater length; edges of crests begin to break into spindrift; foam is blown in well-marked streaks.</td><td>Treble-reefed topsails etc.</td><td>Stormachtig</td><td>7.5</td></tr>
<tr><td>9</td><td>20.8 – 24.4</td><td>42 – 49</td><td>Strong Gale: Slight damage occurs to buildings, shingles are blown off of roofs.</td><td>High waves; sea begins to roll; dense streaks of foam; spray may reduce visibility.</td><td>Close-reefed topsails and courses</td><td>Storm</td><td>9.5</td></tr>
<tr><td>10</td><td>24.5 – 28.4</td><td>49 – 57</td><td>Storm: Large trees are uprooted, building damage is considerable.</td><td>Very high waves with overhanging crest; sea takes white appearance as foam is blown in very dense streaks; rolling is heavy and visibility is reduced.</td><td colspan="2">She should scarcely bear close-reefed main-topsail and reefed fore-sail.</td><td>Zware storm</td><td>12.0</td></tr>
<tr><td>11</td><td>28.5 – 32.6</td><td>57 – 65</td><td>Violent Storm: Extensive widespread damage. These typically occur at sea, and rarely inland.</td><td>Exceptionally high waves; sea covered with white foam patches; visibility still more reduced.</td><td colspan="2">Would reduce her to storm staysails.</td><td>Zeer zware storm</td><td>15.0</td></tr>
<tr><td>12</td><td>>32.7</td><td>>65</td><td>Hurricane: Extreme destruction.</td><td>Air filled with foam; sea completely white with driving spray; visibility greatly reduced.</td><td colspan="2">No canvas would withstand.</td><td>Orkaan</td><td>>15</td></tr>
</table>

Table 3-1 Beaufort scale

4. OCEANOGRAPHY

4.1 Introduction

Oceanography has been studied since 1725, when the Italian Count Luigi Marsigli wrote one of the first books on the subject. Matthew Maury, a United States Naval Officer, wrote the first "modern" oceanography book in 1855. Many of his observations - compiled from ship logs - are excellent; all are interestingly explained, even though he had no knowledge of geophysics.

The first systematic, specific study of the oceans was carried out by the H.M.S. Challenger. This ship sailed from Portsmouth, England on the 21st of December 1872, and in 3½ years she sailed more than 100,000 km. The measurements and observations resulted in a 50-volume report. This was also the first report to subdivide oceanography into its four modern major fields:

- biological oceanography
- chemical oceanography
- geological oceanography
- physical oceanography

In this section, some aspects of physical oceanography are described. However, one must realise that biological, chemical, and geological processes have a major influence on, and are influenced deeply by coastal engineering measures in the marine environment.

The mean depth of the oceans is about 3.8 km (the average depth of the North Sea is 94 m). The shallowest part of the oceans, adjacent to the landmasses is called the continental shelf. This makes up 7.6 % of the total ocean area (Figure 4-1). The continental shelf reaches depths up to 200 meters. The oceans are subdivided into a series of interconnected basins in which most of the interesting physical oceanographic activity takes place. These basins are 3 to 5 km deep with occasional deeper or shallower sections. Most of the interesting processes in the oceans take place in the upper 1 to 2 km. Deeper than this, the oceans are of rather uniform salinity (35‰) and temperature (3° - 4° C). Currents in the deep zone are very weak - often assumed to be zero. In Sections 4.2 and 4.3, processes in the upper zone of the ocean are described.

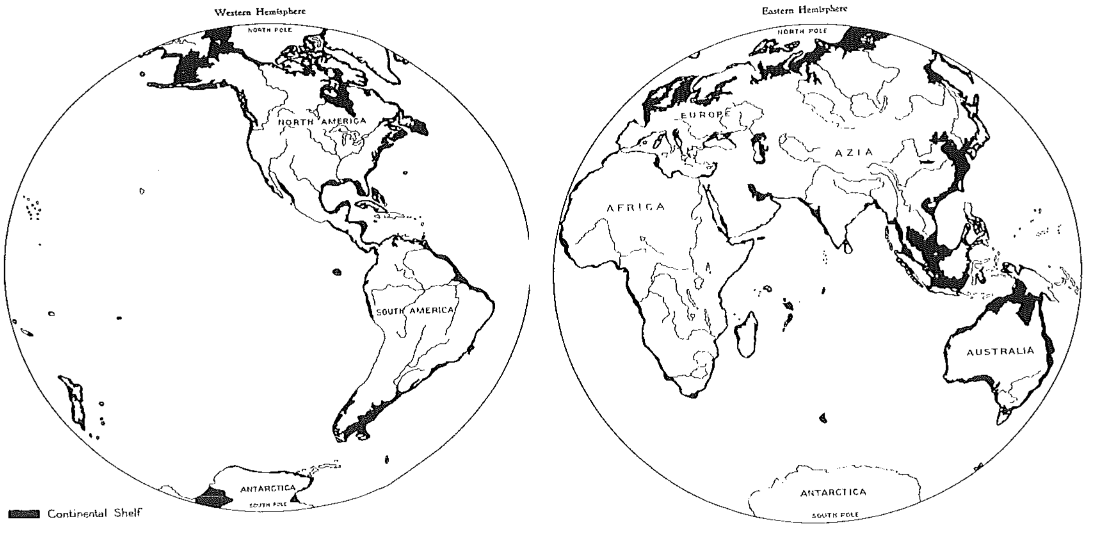

Figure 4-1 Continental shelf

The three primary forces that produce a disturbance of the sea surface are wind (wind waves and probably seiches), earthquakes (tsunamis), and gravitational attraction within the sun, moon and earth system (tides). Tides are described in Section 4.4. Seiches are the subject of Section 4.5; Tsunamis are treated in Section 4.6. In Section 4.7, a short description of short wave theory is given. Only the basic principles are treated in this chapter. Those desiring to learn more details about the subjects treated in this chapter are referred to handbooks or to specific lectures and lecture notes containing a more comprehensive discussion[3].

4.2 Variable density

The density of seawater is a function of three variables:
- salinity
- temperature
- pressure

The influence of pressure on the density can be neglected unless the depth is more than ± 500 m. In contrast to pure water; seawater will continuously increase in density as it cools until it reaches its freezing temperature.

Most seawater has salinity varying between 34 and 36‰. However, some smaller isolated seas can show significant variations: For example, the Baltic Sea sometimes has a salinity as low as 7‰, while the salinity of the Red Sea may be as high as 41‰ .

Salinity and temperature are not constant throughout the water depth. With increasing depth, both salinity and temperature usually decrease. Evaporation is responsible for the higher salinity of the surface layer. A higher salinity means a higher density, whereas a higher temperature causes a lower density. Still, in general, the temperature differences are sufficient to maintain a stable density profile, i.e. where density increases with depth. There are exceptions to this rule, which are important when acoustic surveying is in progress. In areas where an inversion is present, it is possible for objects (submarines) to escape from detection by sonar.

The salinity S is defined as the total amount of solid material in grams contained in 1 kg of seawater, when:
- all the bromine and iodine have been replaced by the equivalent amount of chlorine
- all the carbonate converted to oxide
- all organic matter has been completely oxidised (Forch, Sorensen and Knudsen, 1902)

The salinity defined in this way can be determined with great accuracy.

Chloride compounds ($NaCl$ and $MgCl$) constitute the major portion of the dissolved salts. Therefore, the total salinity S can be derived from the Chlorine content by the formula

$$S = 0.030 + 1.8050\, Cl \tag{4.1}$$

in which Cl is the chlorine content that can be established relatively easily by titration with silver nitrate ($AgNO_3$). Although the relation between S and Cl is not exact, the approximation is adequate for practical engineering purposes. Further details can be found in handbooks on physical oceanography (Defiant, 1961).

There are several methods to measure the salinity of water:

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3 At TU Delft, these specific lectures are: CT3310 (open channel hydraulics), OT3620 (oceanography and waves), CT4320 (short waves), CT5316 (wind waves) and CT5317 (physical oceanography).

- titration with silver nitrate as indicated above
- determination of the density ρ
- measurement of electric conductivity

Calibration of the method used is always important. For this purpose, "standard sea water" can be purchased from well known hydrographic institutes. The samples are stored in sealed glass tubes to prevent evaporation. The density and chlorinity of each sample are given with great accuracy.

Since the density of salt water usually is a little higher than 1000 kg/m^3, oceanographers often subtract 1000 from the density values and denote the value by σ. If this is done for conditions under atmospheric pressure a subscript t is usually added:

$$\sigma_t = \rho - 1000 \quad [\text{kg/m}^3] \tag{4.2}$$

Since the scientific equations and tables to calculate the density are a bit cumbersome in use, WL | Delft Hydraulics uses a simpler relationship for engineering purposes:

$$\sigma_t = 0.75\, S \tag{4.3}$$

where:
S = salinity [in ‰]

This relationship neglects influences of temperature and pressure and is therefore more limited in use. In practice it is sufficient for situations in which density differences result exclusively from salinity differences.

Density variations can be used in ingenious ways. Imagine that we take a long (1km) pipe and put it vertically down from the ocean surface. Next, we attach a pump and slowly draw up the deep water. We do this slowly so that the rising water can be warmed by the surrounding ocean. After water from the depths reaches the surface we remove the pump and find that the water continues to flow. Why? It is not perpetual motion; the process stops as soon as the upper 1-km layer of the ocean has become mixed. The cause for the motion is the lower density of the water in the pipe system. By the slow flow, we even out temperature effects, but in the closed system we prevent evaporation, which causes the higher density near the surface in the free ocean.

This form of energy conversion has been the subject of many research projects. Under the name OTEC (Ocean Thermal Energy Conversion), now part of DOWA (Deep Ocean Water Applications), many countries are trying to develop power plants based on the difference in temperature and the difference in density. Tropical areas are particularly suitable for ocean energy systems, when deep (> 1000m) gullies are found close to the shore. On Hawaii, such a plant has been constructed.

Until now, density differences in the vertical profile have been described. Horizontal density variations can occur, too. For example: in a tidal river mouth, salt water enters the estuary during rising tide (unless there is more than enough fresh water flow in the river to completely fill the entire tidal prism; few rivers have sufficient flow over the entire year to prevent the intrusion of salt water). Accordingly, at some point in a river salinity can be expected to vary according to the tide. Often, a density gradient causes a density current. This depends on the stability of the actual configuration, and will be discussed in one of the following chapters.

4.3 Geostrophic currents

The ocean is not a static body of water. (If it were, then "bottle mail" would never work out, right!) We have seen that there are dominant wind patterns around the globe. They cause friction at the interface with the water and are thus the driving force for very characteristic flow patterns around the globe as well. We can recognise a strong westward current around the equator and eastward currents at latitudes between 40° and 60° in both the N. and S. hemisphere. In the N hemisphere, the circulation cells are moving clockwise, while in the S. hemisphere, they run anti-clockwise. In the North Atlantic, the Gulf Stream, running from Florida to Norway is best known. Similar flows are known near Japan (Kuro-Shio), along the coast of Latin America (Brazil stream, and along the E. coast of Africa the mighty Agulhas drift. The major driving force for these *geostrophic* ocean currents is the prevailing wind at different latitudes. The maximum velocities occur in the upper layer (the upper 500 to 1000m) and are small (< 1m/s). Although the total body of moving water is enormous, bottom friction is relatively unimportant. On the other hand, the Coriolis effect does play a very important role.

The geostrophic currents play a dominant role in the re-distribution of solar energy around the globe. The masses of warm tropical water moving from the equator to the poles constitute about 50% of the heat transfer between the tropical and the polar zones. The other 50% is transferred via the atmosphere.

The current patterns are not completely static. Short-term variations that are related to short term climatic variations (El Niño) are observed. It is suggested by some sources that long term variations in the climate are related to long term variations of the geostrophic current patterns. That makes observation of these patterns extremely relevant.

The considerations so far, have been limited to the surface currents. Oceanographic expeditions have demonstrated that even in tropical areas, deep ocean water is very cold. In Polar Regions, the density of seawater increases due to the lower temperature. When the sea surface freezes, the salt content of the remaining water increases (ice contains no salt). This also adds to the density of the seawater in Polar Regions. This high density leads to sinks in the polar zones. The water carried by the Gulf Stream to the Norwegian coast attains an ever-increasing density, not only due to the lower temperatures, but also due to a strong evaporation. The entire mass of water does not turn southward as a surface current along the Norwegian coast. Part of it submerges due to the high density and flows southward at a much greater depth, thus contributing to the low temperature in the deeper parts of the ocean. This is called the thermohaline circulation. The cold undertow flows southward through the Atlantic Ocean, along Antarctica and eventually surfaces in the Pacific Ocean. There, it forces warmer and lighter water to return as a surface current moving via the Indonesian Archipelago around Cape of Good Hope to the Atlantic system. Another part flows via Cape Horn back into the Atlantic Ocean. It is estimated that the circulation period is in the order of 1000 years. Gordon and Broecker have familiarised the theory under the name “Ocean Conveyor Belt”. (See Figure 4-2).

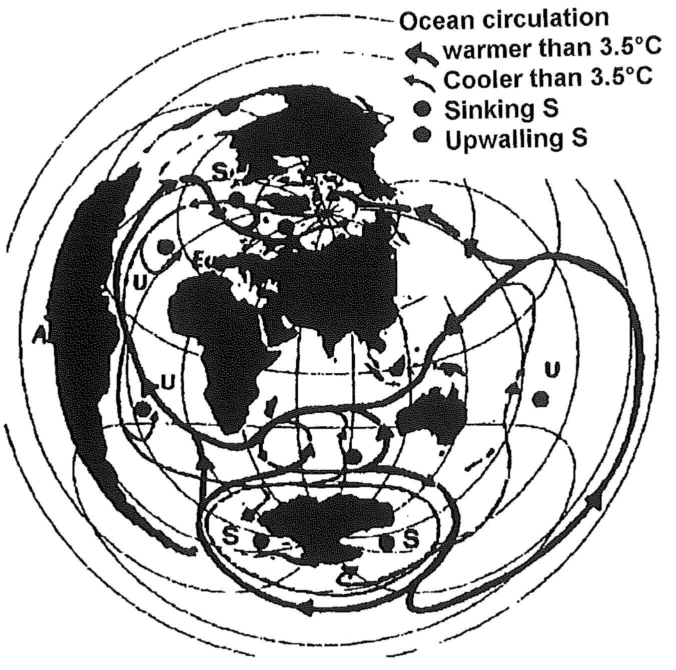


Figure 4-2 Ocean conveyor belt [A.L. Gordon, 1986]

4.3.1 Coriolis Acceleration

In all considerations about air or water currents around the globe, we are confronted with the fact that Newton's equations of motion (i.e. displacements, velocities and accelerations) refer to a fixed grid, whereas our observations of velocity and acceleration refer to a grid system attached to the moving and rotating earth. This system is not at rest as meant by Newton when he formulated his equations of motion. The deviations arising because we speak about relative movements instead of absolute movements were investigated by Coriolis (1792-1843). In brief, the Coriolis effect causes a free current in the Northern Hemisphere to turn slightly to the right, and in the Southern Hemisphere to the left. It is complicated to visualise the cause of the Coriolis effect in a simple physical manner. For a proper understanding one needs the explanation via a comprehensive analysis of the theory of relative motion, and in particular the theory applied to rotating vehicles. This theory is treated in a very clear manner by Den Hartog (1948). The relevant chapters from his work have been added as Annex 2.

The deviation from the straight path can be quantified by the introduction of the Coriolis acceleration:

$$a_c = 2\,\omega_e\,V\,\sin\phi \qquad (4.4)$$

where:

a_C = Coriolis acceleration
ω_e = angular velocity of the earth = 72.9 * 10^{-6} rad/s (based on sidereal day)
V = current velocity
ϕ = latitude

The acceleration acts towards the right in the Northern Hemisphere and to the left in the Southern Hemisphere. The influence of latitude is due to the fact that all forces have to be split into a component parallel to the local earth surface and a normal component that works along the same vector as gravity.

If the flow takes place in a confined conduit or channel that prevents a deviation of the course (i.e. a steady current), the Coriolis acceleration causes a pressure gradient across the conduit:

$$\frac{1}{\rho}\frac{\partial p}{\partial n} = 2\,\omega_e\, V \sin\phi \tag{4.5}$$

where:
ρ = water density
p = water pressure
n = normal to the current

In open channel flow, the pressure gradient becomes visible as a gradient of the water surface. This can be illustrated by computing the sea level difference across the Strait of Florida, the result is 0.52 m (elevation difference).

The Florida Current is located at latitude 26 °N; the current velocity is about 1.0 m/s; the width of the Strait of Florida is about 80 km.

$$\frac{1}{\rho}\frac{\partial p}{\partial n} = 2\times 0.729 * 10^{-4} \times \sin 26° \times 1.0 = 6.4 * 10^{-5}\,\frac{m}{s^2} \tag{4.6}$$

The elevation difference over 80 km is computed as follows:

$$\Delta z = 6.4 * \frac{10^{-5}}{9.81} \times 80 * 10^{3} = 0.52 * 10^{-2}\, m \tag{4.7}$$

The observed value is 0.45 m, which is very close. (Similar computations can be made for the Western Scheldt, the British Channel, or the Zeegat van Texel)

4.4 The tide

4.4.1 The vertical tide

The Newtonian laws apply to the movement of celestial bodies and their mutual influences. The planet earth is part of the solar system, and within this system, it forms a close relation with the sun on one hand and with the moon on the other hand.

Sun and earth rotate around a common centre of gravity. They attract each other by a force that is proportional to the masses and inversely proportional to the square of their distance. The rotation around the common centre of gravity causes a centrifugal force that is equal to the attracting force. A similar balance exists between the earth and the moon. The difference between the two systems is that in the case of the sun-earth relation, the mass of the sun is dominating, and in the case of the earth-moon interaction the mass of the earth is dominating. Comparing the mutual forces in the two systems, one may state that they are of the same order of magnitude (the lunar influence slightly (factor 4) larger than the solar influence). The larger mass of the sun is compensated by its greater distance from the earth. The difference in mass between sun and moon causes the gravitation centre of the solar system to be located inside the sun, and the centre of the lunar system within the earth. In a simplification, we can therefore say that the earth circles around the sun and that the moon circles around the earth.

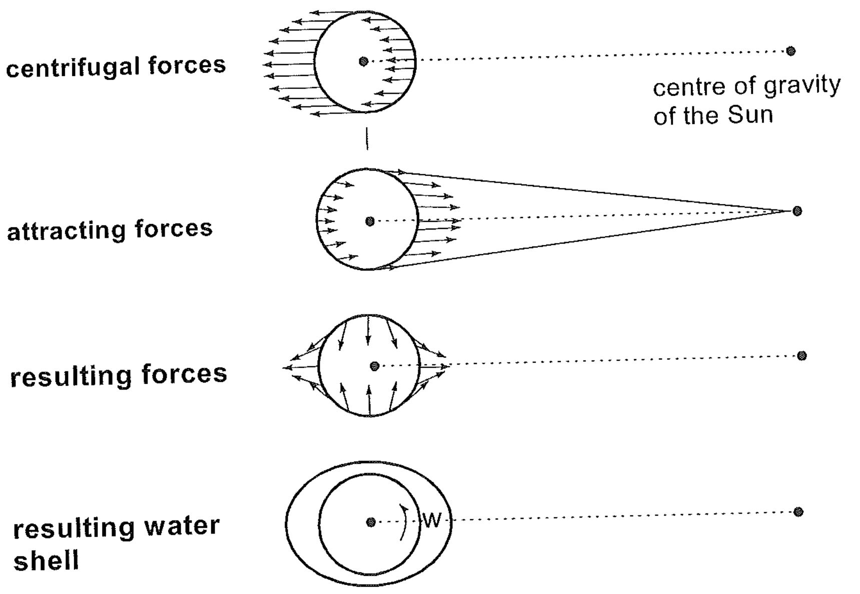


Figure 4-3 Centrifugal and attracting forces of the Earth-Sun system

The main facts about the two systems are (approximate figures):

- The mass of the sun is: 1.99×10^{30} kg
- The mass of the earth is: 5.98×10^{24} kg
- The mass of the moon is: 7.35×10^{22} kg
- The distance between sun and earth is 150.10^{6} km
- The distance between earth and moon is 400 000 km
- The earth circles the sun in 365 days
- The earth rotates around its own axis in 24 hours
- The moon circles the earth in 29 days

The forces described in the foregoing not only apply to the earth as a whole, but also work on individual mass elements on earth so water particles in the oceans are also subject to gravitational and centrifugal forces of both the solar and the lunar systems. The gravitational forces work in one direction, the centrifugal forces in the opposite direction.

If the earth were completely covered by water, its equilibrium configuration would be an ellipsoid (Figure 4-4). As the earth also turns around its own axis, an observer could see two high and two low waters passing every day. This is true for the solar influence. When considering the lunar influence, one must realise that the next day, the moon has changed position (1/29th of the total orbit around the earth); the water ellipsoid has turned, too. Therefore, the period of the lunar tide is 12 hours and 25 minutes.

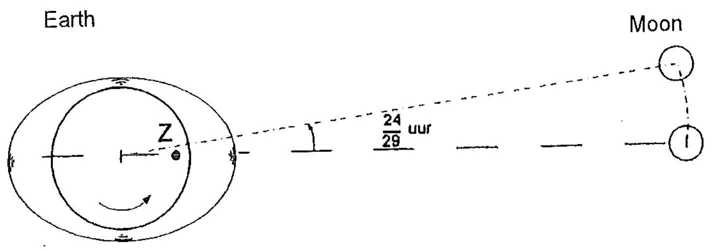


Figure 4-4 Equilibrium (moon) tide

When sun, earth and moon are in one line (full and new moon), the solar and lunar tides reinforce each other. The ellipsoid becomes more pronounced and the tide gets a bigger amplitude and is called *spring tide*. When the solar and lunar tide are 90° out of phase, their effects cancel each other (first and last quarter). The ellipsoid approaches a circle, and consequently the tide gets a smaller amplitude. This situation is called *neap tide* (Figure 4-5).

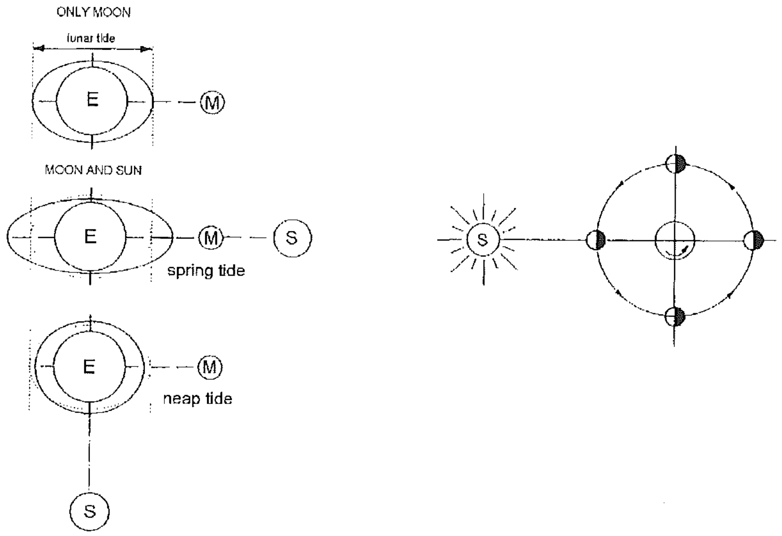


Figure 4-5 Spring and neap tide

So far, the earth was schematised as if it were completely covered with water. In reality, continents prevent the development of the tidal ellipsoid. In the Southern Hemisphere, however, the original ellipsoid can develop; there, the tidal wave travels around the earth at a latitude of about 65° S, south of Africa, South America and Australia. From there, the tidal wave propagates to the North into the Atlantic, Indian and Pacific Oceans.

The tidal wave is a long wave ($L >> d$). The water motion can be expressed by the long wave equation in two dimensions. If friction can be neglected, the wave propagation speed is:

$$c = \sqrt{gd} \tag{4.8}$$

where:
c = wave propagation speed [m/s]
g = gravitation acceleration [m/s^2]
d = water depth [m]

Assuming an average depth in the Ocean of 4000m, the propagation speed of the tidal wave is about 200m/s, or over 750 km/h! In this way, it takes the tidal wave up to several days to reach the most remote spots in the N. hemisphere. On its way, the tidal wave is distorted by the non-uniform depth contours of the oceans and coastal waters and, since the propagation of the wave involves the motion of water, it is also influenced by the Coriolis acceleration. It is possible to visualise the propagation of the tidal wave by mapping the lines of simultaneous HW (in sun hours after moon culmination) and the lines of equal tidal range (vertical distance between HW and LW in m). At some locations, the amplitude of the vertical tide can become zero. This is called an amphidromic point. This is illustrated in Figure 4-6 and Figure 4-7.

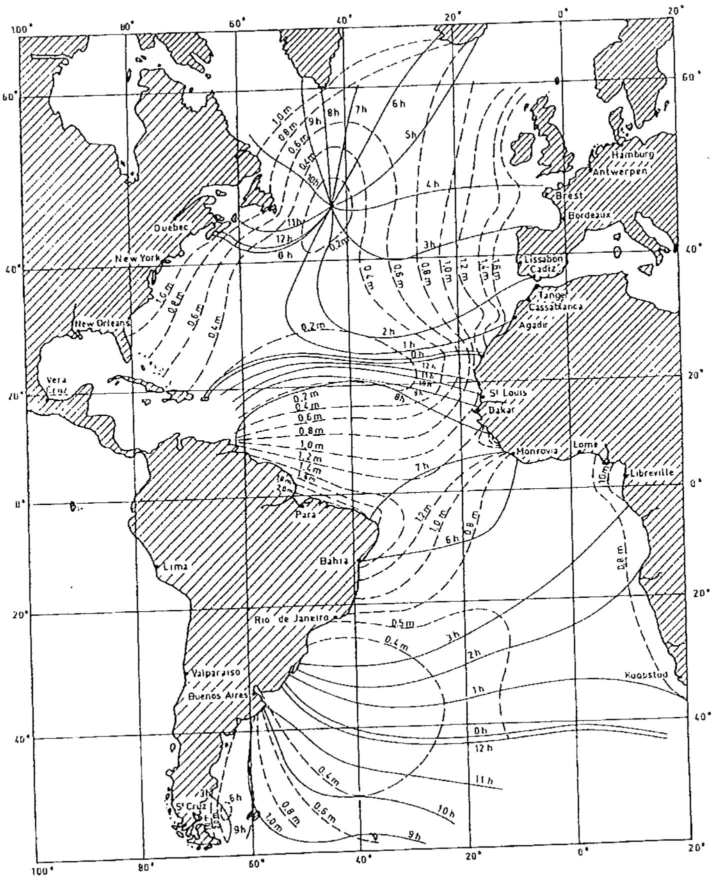


Figure 4-6 Propagation of the tide in the Atlantic Ocean

Thus, every place along the coasts of the world has its own specific tidal curve. At some locations, the difference between high and low water is up to 12 m. The further the location is away from the South Pole, the longer is the time shift between the celestial event and its appearance in the form of the tide. In this way, in the Netherlands, the occurrence of spring tide and neap tide is about two days after the corresponding moon configurations.

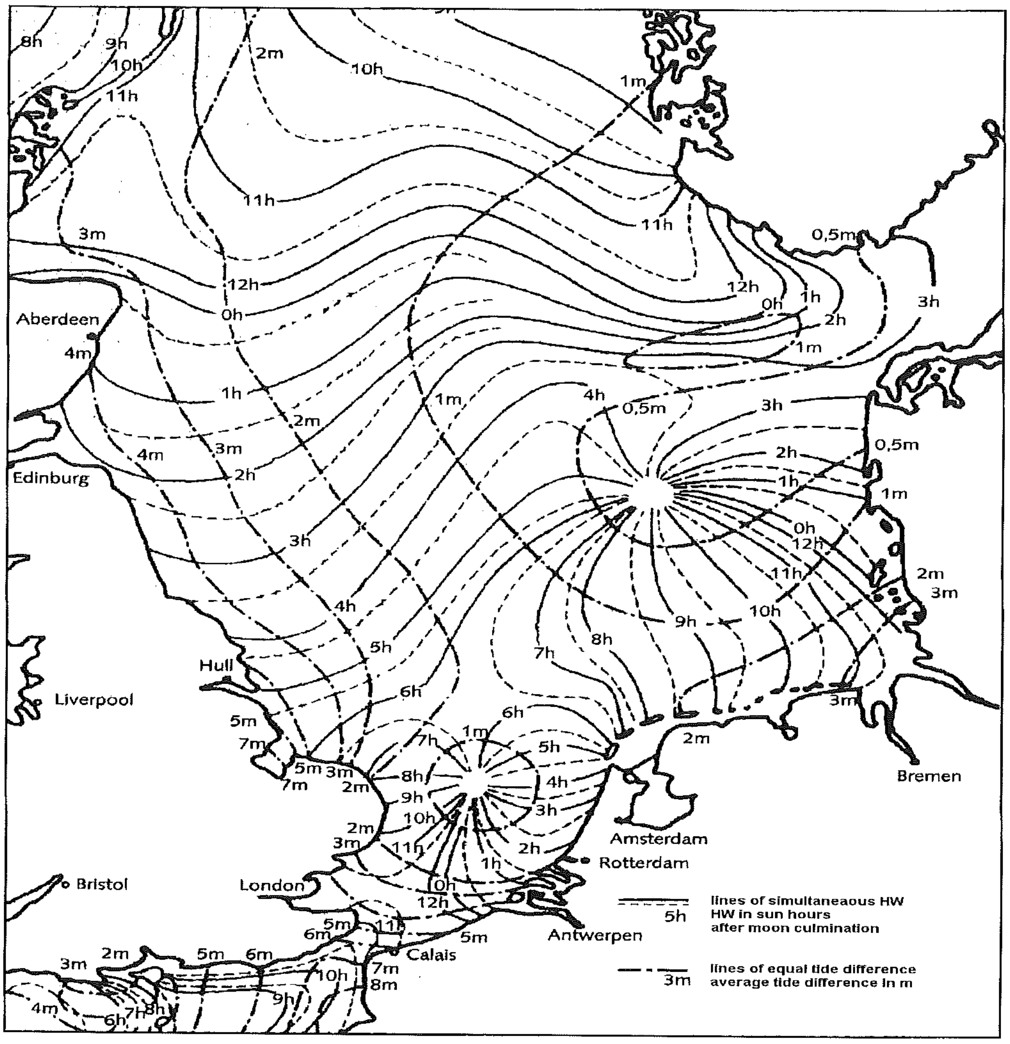


Figure 4-7 Propagation of the tide in the North Sea

So far celestial influences have been simplified to the direct gravitational influences of sun and moon. Other phenomena also play a role and of these the declination is one that cannot be overlooked. The earth's axis (from North to South Pole) is not perpendicular to the earth-moon connection line (Figure 4-8). The angle between the equatorial plane and the earth-moon line is called declination, δ. Following from that, the two high and low waters on a day are not equal. At some latitudes, this daily inequality becomes so big, that there is only one high and one low water.

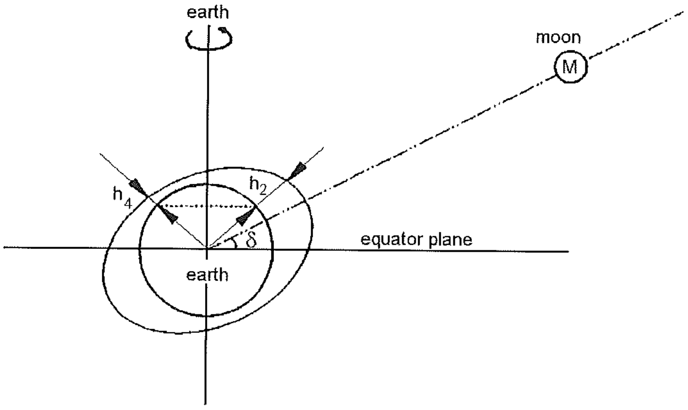


Figure 4-8 Daily inequality of the lunar tide

Because the tide is caused by regular astronomical phenomena, it can be predicted accurately a long time ahead (although not including meteorological effects). The method used for tide prediction is called harmonic analysis. The water level at a certain location as a function of time is expressed by the following formula:

$$h(t) = h_0 + \sum_{i=1}^{N} h_i \cos(\omega_i t - \alpha_i) \tag{4.9}$$

where:

$h(t)$ = measured (or predicted) tidal level with reference to a fixed level (m)
h_0 = mean level (m)
h_i = amplitude of component number i (m)
ω_i = angular velocity of component number i (1/h)
α_i = phase angle (-) of component number i (-)
t = time (h)

Contrary to the traditional harmonic analysis, the frequencies ω_i are known here, having been derived from astronomical considerations. The phase angles α_i have to be derived from observations as they are extremely site specific. This applies to the amplitude h_i as well.

Each component has an internationally agreed abbreviation. The influence of the sun is characterised by the letter *S*, the influence of the moon by the letter *M*. The index 2 refers to phenomena that occur twice daily, the so-called semi-diurnal effects. M_2 and S_2 thus represent the most common tidal components. Where, due to the daily inequality, only one HW and one LW per day appear, there is a diurnal tide. Diurnal components carry a subscript 1. Higher order components carry a subscript 3, 4 or higher.

An example of the result of a harmonic analysis for some ports along the Dutch coast is presented in Table 4-1 . This table shows the main harmonic components used for prediction of the astronomic tide. Nota bene: In the Table A_0 gives the difference between NAP and MSL. Close to the coast this difference can be neglected; but if one looks at a river farther upstream, the river gradient influences the mean sea level. The mean level A_0 changes a little during the year, as can be seen from the small amplitude of SA. The angular velocity of this component (0.041) leads to a period of 365 days.

Contrary to the Dutch tide tables, in other such tables A_0 represents the difference between a Chart Datum and Mean Sea Level. Chart Datum is then defined as a low level that is exceeded rarely, for instance LLWS. Because of this site-specific definition of the Datum level, it is not necessary for the Datum plane to be horizontal. Utmost care is required when performing hydraulic calculations in this case. (See also Appendix 5). The Datum level used by different countries for the same waterway can also be different, which leads to different depth figures for the same location. This is the case for the Western Scheldt, where Dutch and Belgian charts show such differences.

Component	Angular Velocity In ° per hour	Amplitude cm Phase lag g° ref. to MET	Vlissingen 51^0-27^1 N 3^0-36^1 E	Euro Platform 52^0-00^1 N 3^0-17^1 E	H. of Holland 51^0-59^1 N 4^0-07^1 E	Rotterdam 51^0-55^1 N 4^0-30^1 E	IJmuiden 52^0-28^1 N 4^0-35^1 E	Delfzijl 53^0-20^1 N 6^0-56^1 E
A_0	ref. to NAP in cm		-1	0	7	24	2	7
SA	0.041	H cm	7	9	8	7	10	9
		g^0	216	213	222	241	220	219
SM	1.016	H cm	4	3	3	3	3	4
		g^0	33	31	32	43	22	32
Q_1	13.399	H cm	3	4	3	3	4	3
		g^0	133	126	131	148	133	179
O_1	13.943	H cm	11	11	11	9	11	9
		g^0	195	188	191	209	193	247
P_1	14.959	H cm	3	3	3	2	3	3
		g^0	353	340	346	11	346	48
K_1	15.041	H cm	7	8	8	6	8	8
		g^0	10	358	359	17	358	43
$3MS_2$	26.952	H cm	3	2	2	2	2	4
		g^0	281	288	312	344	338	167
MNS_2	27.424	H cm	3	1	2	2	2	3
		g^0	143	154	182	211	210	33
NLK_2	27.886	H cm	4	2	2	2	2	4
		g^0	354	1	26	58	54	245
μ_2	27.968	H cm	13	6	8	8	9	15
		g^0	161	174	200	232	227	55
N_2	28.440	H cm	29	12	12	10	10	21
		g^0	35	26	59	95	108	310
NU_2	28.513	H cm	9	4	5	5	4	8
		g^0	26	25	52	86	88	288
MPS_2	28.943	H cm	3	1	1	1	2	5
		g^0	110	107	170	206	205	27
M_2	28.984	H cm	175	74	79	72	68	136
		g^0	59	54	86	121	129	333
λ_2	29.456	H cm	6	3	3	3	3	5
		g^0	76	80	110	144	142	348
$2MN_2$	29.528	H cm	13	6	7	7	7	12
		g^0	257	261	290	325	323	168
S_2	30.000	H cm	48	18	19	17	17	34
		g^0	117	111	147	184	198	46
K_2	30.082	H cm	14	5	6	5	5	10
		g^0	117	111	147	184	198	43
$2S M_2$	31.016	H cm	4	2	2	2	3	4
		g^0	348	358	25	61	54	270
$2M K_3$	42.927	H cm	3	1	1	1	1	1
		g^0	162	141	191	225	263	120
MK_3	44.025	H cm	2	1	1	1	0	1
		g^0	316	281	288	349	279	278
$3MS_4$	56.952	H cm	2	1	2	2	3	4
		g^0	196	193	235	303	268	216
MN_4	57.424	H cm	4	4	6	5	7	5
		g^0	94	105	137	204	157	118
M_4	57.968	H cm	13	10	17	15	20	17
		g^0	120	130	165	230	186	145
MS_4	58.984	H cm	9	7	11	9	12	10
		g^0	181	185	222	291	246	224
MK_4	59.066	H cm	2	2	3	3	4	3
		g^0	178	184	221	290	244	222
$2MN_6$	86.408	H cm	5	2	2	2	2	4
		g^0	82	64	95	211	269	321
M_6	86.952	H cm	9	4	5	4	4	7
		g^0	109	92	128	243	290	352
$2MS_6$	87.968	H cm	9	4	4	4	5	7
		g^0	161	146	188	302	343	61
M_8	115.936	H cm	3	1	2	1	3	1
		g^0	115	142	230	358	330	217
$3MS_8$	116.952	H cm	5	2	4	2	4	2
		g^0	166	194	281	51	23	276

Table 4-1 Main constituents of the tide at several places in the Netherlands

4.4.2 The horizontal tide

The variation of the water level is called vertical tide. The currents resulting from this variation are called horizontal tide. These currents can be calculated using

- the driving force of the vertical tide
- the friction
- the storage capacity of the area concerned

Depending on the local conditions, friction or storage can be simplified or neglected. Calculations of this kind are dominated by the boundary conditions imposed. It is necessary to select boundaries carefully, far enough from the area of interest to eliminate model distortions, and possibly at a place where they can be simplified to merely level variation or flow variation. If the model is used to study the effect of changes in the physical surroundings (dredging, dam construction), the boundaries must be chosen at such a distance that they are not affected by the changes.

Because of the uncertainty about storage capacity and friction, calculations of tidal currents must be calibrated by flow measurements. Such measurements must last at least a full tidal cycle (12 hours 25 min.) and must preferably be done from slack water to the next slack water. (Slack water need not coincide with HW or LW). It is specifically recommended that current measurements should be carried out during both regimes when the difference between neap tide and spring tide is considerable,.

When considering horizontal tides in a river mouth, one must also take into account the influence of the upland discharge.

4.4.3 Special Effects

Some rivers located at the landward end of an estuary experience another extreme tide-dependent condition - a tidal bore, an abrupt and migrating rise in the water level at the beginning of the flood tide (Figure 4-9). This "wall of water" is a response to the quick reversal from an ebbing tidal condition to a flooding one. Bores are uncommon, forming only in special circumstances that depend on tidal conditions and the morphology of the estuary. The bore in the Truro River of the Bay of Fundy is typically only about half a metre high. In the Bay of St. Malo on the northern coast of France, a bay with the world's second largest tidal range, the bore rarely exceeds a meter in height. Large tidal bores occur in the Pororoca River, a branch of the Amazon, and in the Chien-tang estuary in China. The bore reaches 5 m in the Pororoca and nearly that height in the Chien-tang.

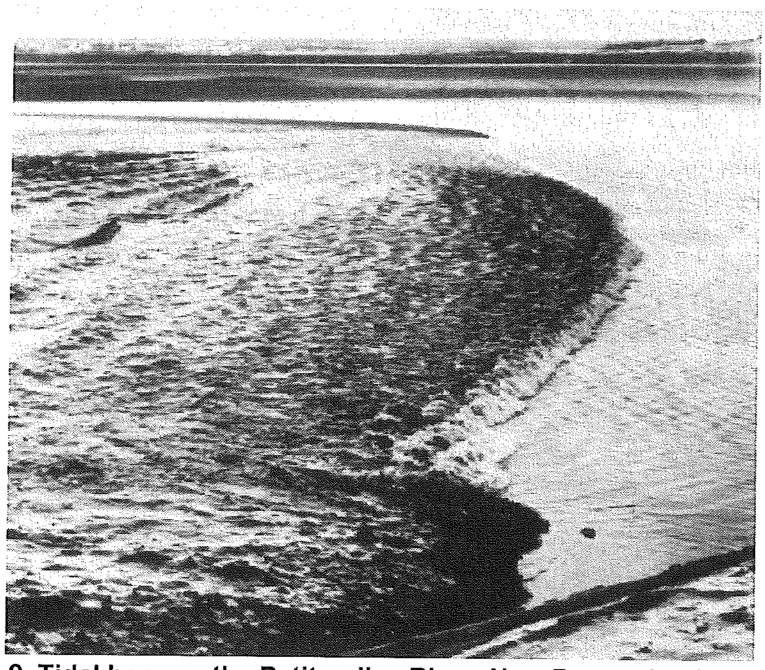


Figure 4-9 Tidal bore on the Petitcodiac River, New Brunswick (Stowe, 1987)

4.4.4 Deviations from the astronomical prediction

So far, water level variations along the coast have only been attributed to the influence of tides. It has been pointed out that the astronomical tides can be predicted accurately long in advance.

The water level in the vicinity of the coast, or even in an estuary or bay, is influenced by more parameters than the astronomical tide alone. Parameters that play an additional role are:

- Atmospheric pressure differences
- Wind shear over the water surface
- Coriolis acceleration
- Rainfall and river discharge
- Surface waves and associated wave set-up
- Storm motion effects

The influence of atmospheric pressures can be understood by considering the water level at sea as being contained in one large communicating vessel. A relative drop in air pressure in a particular region will result in a corresponding rise of the sea level and vice versa, taking into account the difference between the density of air and water.

Wind shear stresses over the water surface generate a flow of water, which in its turn causes a gradient of the water level $i = \Delta h/\Delta s$. The gravity force generated by this gradient must eventually compensate the shear force that works in the opposite direction. This leads to the well-known empirical formula for wind set-up in a closed basin:

$$i = c\frac{U^2}{gh} \tag{4.10}$$

in which

i = gradient of the water surface
U = wind speed in m/s
g = acceleration of gravity in m/s^2
h = water depth in m
c = empirical coefficient (3.5 to 4 * 10^{-6})

The formula clearly indicates that this effect is important in shallow waters, i.e. in areas with an extensive and shallow continental shelf.

Rainfall plays a role when an enclosed area is considered and the quantity of rainfall is excessive. This can be the case in a tropical hurricane. River discharge plays a role in estuaries and river mouths. The discharge can be caused by an independent event or by the same event that causes atmospheric depression and storm.

Wave set-up by surface waves is caused by the transformation of kinetic energy into potential energy. It is not treated here in detail. The reader is referred to specialised literature on short waves.

Most of these parameters exert their influence in relation with the topography, that is the layout of the coastline and the depth contours in front of it.

It is evident that most of the additional parameters mentioned here are closely related to the prevailing weather conditions. Since it is still not feasible to give accurate weather predictions over a longer period, it will be clear that the prediction of the actual water level along the coast for a given moment or a given period is scarcely possible. Therefore the character of each of the parameters in itself is stochastic, and this certainly applies for their combined effect. Therefore extreme water levels can not be predicted long in advance, their occurrence must be treated statistically.

In the Netherlands, we are fortunate enough to have a series of observations of extreme water levels that covers almost two centuries. For a proper statistical evaluation (subsequent observations must be independent of each other) only the annual maximum levels are taken into account. This leads to the well-known probability curve for extreme sea levels along the North Sea (Figure 4-10).

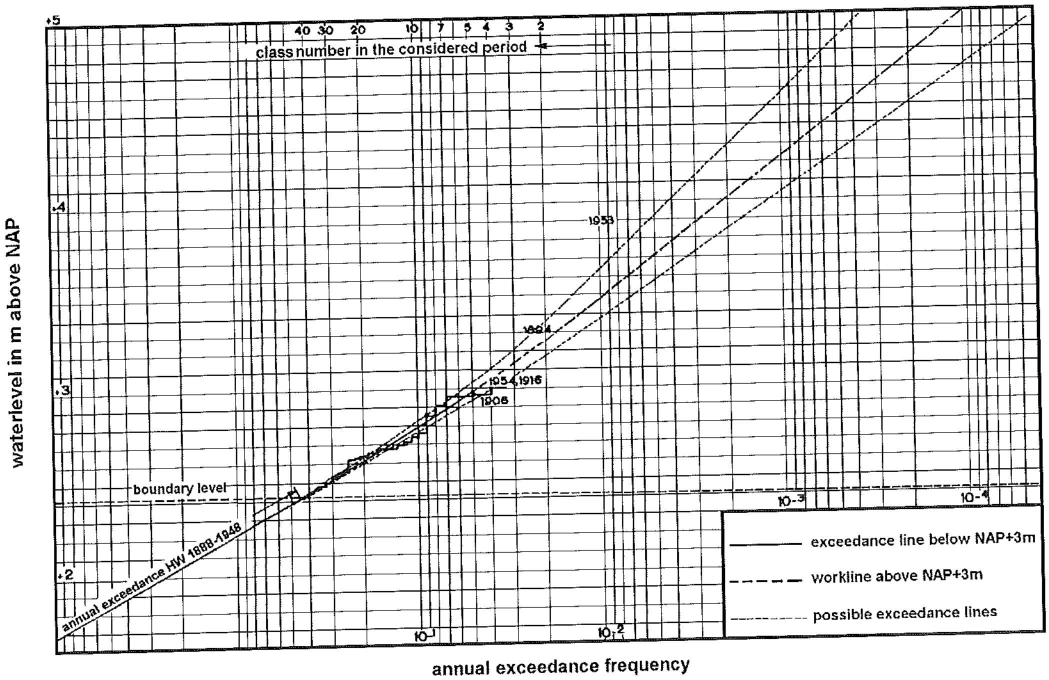


Figure 4-10 Probability curve for extreme sea levels, Hoek van Holland 1859 / 1958 from: Final Report of the Delta Committee, Part I

In other countries, it may be possible to derive similar statistics from the occurrence, strength and path of hurricanes, cyclones or other storm events.

The availability of statistical data on extreme water levels is very important for the design of structures along the coast, ranging from breakwaters, dikes and seawalls to jetties. The occurrence of extreme water levels is also important with regard to the behaviour of natural systems like dunes and sandbars. The shape of a sandy coast may change completely under the combined influence of a high Still Water Level and high waves.

Although the occurrence of extreme water levels cannot be predicted long in advance, the availability of a warning system with a short lead time can be very important in relation to the mitigation of flood damage, and certainly to the reduction of the loss of lives, (provided adequate evacuation facilities are present).

4.4.5 Sources of information

It is possible that a civil engineer needs tidal data for a certain location. The first and most important source is the tide table published by the local authorities. Information on global tide predictions and tidal constants can best be obtained from the Admiralty Tide Tables, edited by the British Hydrographic Department. One must realise that a tidal curve is strictly bound to a certain location. Along coasts, big differences can occur even within short distances. Where no acceptable tidal prediction is readily available the engineer can make a reasonable prediction himself, based on a limited period of hourly observations during 28 consecutive days.

Tidal observations cannot be made without the use of instruments, ranging from a simple tide pole to an automatic recording gauge. The first reliable tide gauge was invented in 1882 by Sir William Thomson, later Lord Kelvin, a Scottish physicist. It consists of a float inside an open pipe attached to a pier. The top of the pipe extends from near the floor of the seabed to above HW level. The base of the pipe is above the bottom; only the slow rise and fall of the tide invades the pipe. A pen records the movement of the float on a graph-paper covered cylinder that is driven at a constant speed. Nowadays most stations have more modern electronic recorders that automatically transmit digital information to a computer.

4.5 Seiches

Between long periodic waves, like the tide, and short waves, like wind waves, a category of waves exists with periods ranging from 100 to 10 000 s. Although their amplitude in the open sea may be small, they can be amplified by resonance, for instance in harbour basins. This effect is called seiche. It was first observed as a standing wave in a mountain lake, from which the name seiche is derived. Driving forces can be pressure variations, discharge variations, tidal influences and swell. Seiches can cause havoc in a harbour by setting up reversing currents at the entrance or by rocking ships free of their moorings. They can also abruptly surge onto piers and beaches and sweep people away. The Great Lakes of North America and some of the large lakes in Switzerland are especially prone to seiches, because they are enclosed basins with large fetches and strong winds.

Figure 4-11 shows an example of a standing wave in a lake, Figure 4-12 shows the possible amplification in a semi-enclosed body of water.

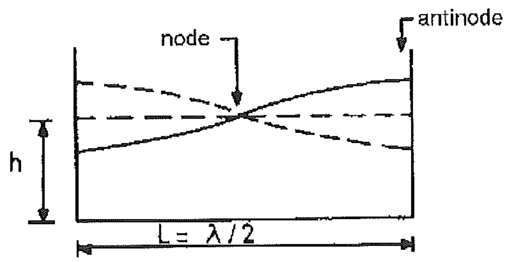


Figure 4-11 Standing wave in a closed body of water

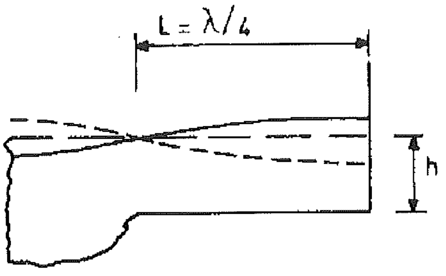


Figure 4-12 Standing wave in a semi-enclosed body of water

In simple cases the wavelength is twice or four times the basin length, but other possibilities exist:

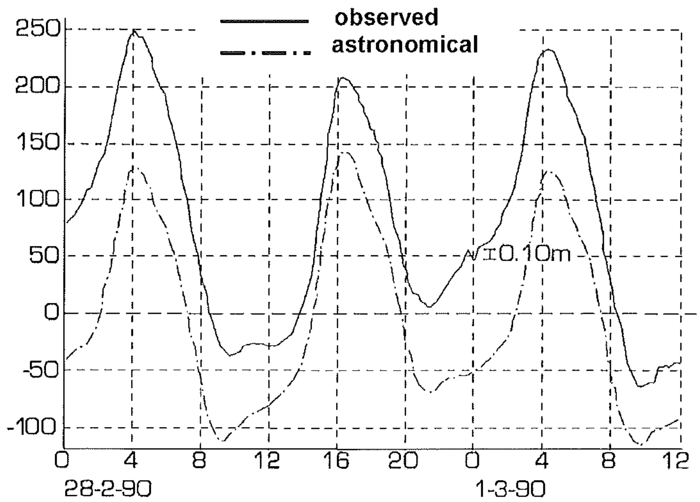
$$T_i = \frac{4\,L_b}{i\sqrt{gh}} \tag{4.11}$$

where:
T_i = period of wave configuration number i
L_b = length of the basin
i = wave configuration number (harmony number)

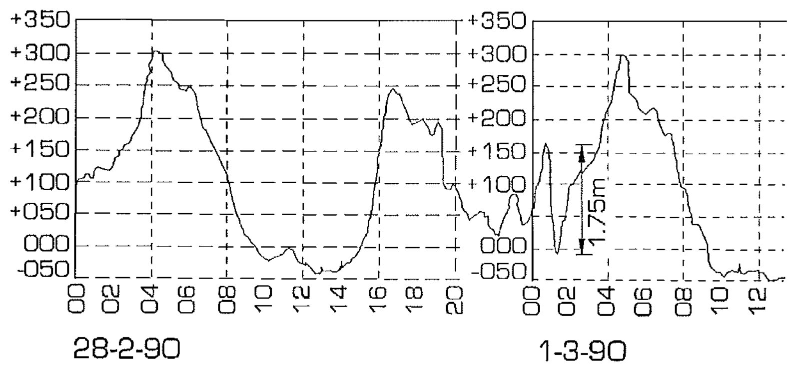
Usually, the vertical amplitude of a seiche, even at an antinode, is small. However, especially at a node, the horizontal displacement of the water can be significant. This can cause mooring difficulties for ships. Another related influence on large ships is the effect of the water surface slope.

As seiches are a resonance phenomenon, it is obvious that the basin size in relation to the wavelength is an important factor. Therefore, measures against generation of seiches are usually based on size restrictions of harbour and other basins, and on the use of irregularly shaped basins.

In the Port of Rotterdam, a seiche was observed in the morning of 1st March 1990. It appears first as a minor fluctuation of around 10cm at Lichteiland Goeree (an observation post some kilometres offshore) at 0.00 hours. Then, at about 01.30 hours, it appears as a huge standing wave of 1.75 m at Rozenburgse Sluis, a navigation lock some 15 km inland. (See Figure 4-13)



Light Island Goeree



Rozenburg Lock

Figure 4-13 Seiche in the port of Rotterdam

4.6 Tsunamis

A tsunami, also called seismic wave, originates when a forceful earthquake or landslide suddenly shifts or displaces a large amount of seawater and sets a train of waves in motion on the sea surface. It moves at great speed, depending on the depth ($c = \sqrt{gh}$) which, for example in 4000 m of water amounts to 200 m/s (700 km/hr). Its length depends on the period ($L=cT$). If T=10 sec, L=2km. In deep water a tsunami often passes unnoticed because it is not so high in deep water. As it approaches the continental coast, it already begins to slow down and steepen in relatively deep water (at depths of hundreds of meters) because of its enormous wavelength. This steepening can begin as far out as 50 km offshore, and the wave that finally hits the coast is huge and forceful, possibly high as 25 m - a prescription for disaster.

The most life-destroying tsunami on record appears to have hit Awa, Japan, in 1703. It killed more than 100,000 people. Among the uncounted victims of the Lisbon earthquake of 1755 were those drowned by tsunami waves in Lisbon and in nearby coastal villages of Portugal and Spain. The trough of the tsunami arrived first, drawing water out of the bay and exposing the sea floor. (Recall that water in a wave moves backward in the trough). Among the drowned were those who came to see the strange sight of the receding waters and were swept away when the crest arrived. Unfortunately, the same situation has recurred numerous times in association with this phenomenon.

In Figure 4-14, the situation after the 1983 tsunami in Minehaha is shown. The tsunami lifted large fishing boats about 6 to 8 m above sea level. After the 1946 tsunami, seismologists began work on a seismic sea-wave warning system. By the early 1960s, a network of seismic monitoring stations covered the entire Pacific Ocean, the only basin where strong earthquakes are common. Knowing the location of the earthquake, seismologists can now predict the path and rate of tsunami movement and provide warnings for most areas, thereby allowing at least a few hours of preparation time before the waves hit a given coast. Generally this is enough time to evacuate people. Although coasts near the origin of the earthquake may receive as little as 10 to 15 minutes advance notice, loss of life has been greatly reduced since the system came into effect.

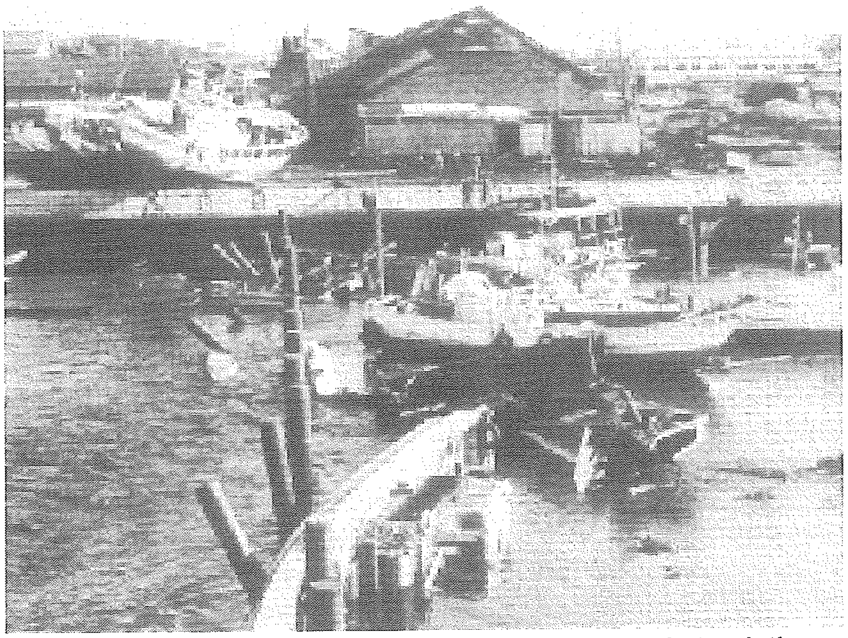


Figure 4-14 Situation in Minehaha (Japan) after tsunami struck the coast

4.7 Waves

4.7.1 (Linear) Wave Theory

Surface Elevation

In textbooks the treatment of waves usually starts with a description of linear wave theory. Although this theory is sometimes quite far from reality, it does help the student to acquire insight the basics principles. Assuming that the wave height is small with reference to wave length and water depth, it is possible to describe the pressure field and the flow field, and to analyse changes in the behaviour of waves when they travel from deep water into shallower water. It should also be noted that such theory refers only to monochromatic, regular waves. It is therefore possible to distinguish the following basic elements (See Figure 4-15):

H = wave height in m
T = wave period in s
L = wave length in m
h = water depth in m (i.e. bottom at $-h$)

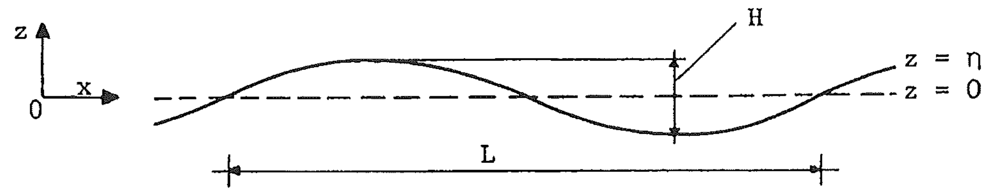


Figure 4-15 Definitions of a regular wave

The application of the linear wave theory leads to a well-known set of equations, the most important of which are given below:

Surface elevation:

$$\eta = \frac{H}{2}\cos\left(\frac{2\pi x}{L} - \frac{2\pi t}{T}\right) \tag{4.12}$$

Or if $2\pi/T$ is substituted by ω, and $2\pi/L$ by k,

$$\eta = \frac{H}{2}\cos(kx - \omega t) = \frac{H}{2}\cos\theta \tag{4.13}$$

Wave celerity:

$$c = \frac{L}{T} \tag{4.14}$$

and

$$c = \sqrt{\frac{gL}{2\pi}\tanh\left(\frac{2\pi h}{L}\right)} \tag{4.15}$$

Wave length:

$$L = \frac{gT^2}{2\pi}\tanh\left(\frac{2\pi h}{L}\right) \tag{4.16}$$

Owing to the specific properties of the hyperbolic functions, it is possible to simplify the above formulas into approximations that are valid for deep (h/L > 1/2) and shallow (h/L < 1/25) water (See Figure 4-16). It must be noted that the limits for shallow water and deep water are approximations in themselves that have no absolute meaning. The values of h/L indicated here have been chosen in such a way that the errors in calculating the wave parameters remain within reasonable proportions for "engineering purpose".

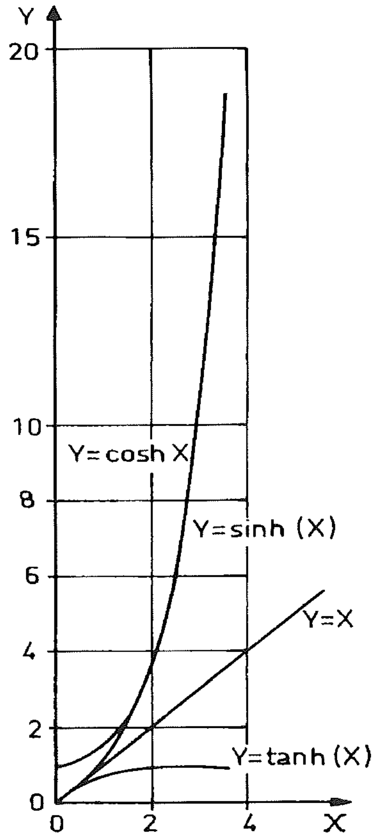


Figure 4-16 Behaviour of hyperbolic functions

For deep water (subscript $_0$), the following applies, because tanh(∞) = 1:

$$L_0 = \frac{gT^2}{2\pi} = 1.56T^2 \tag{4.17}$$

and

$$c_0 = \frac{gT}{2\pi} = 1.56T \tag{4.18}$$

For shallow water, the wave celerity becomes independent of the wave period, because tanh(kh) = kh:

$$c = \sqrt{gh} \tag{4.19}$$

A full list with approximations is given in Table 4-2.

relative depth	shallow water $\frac{d}{L} < \frac{1}{25}$	transitional water $\frac{1}{25} < \frac{d}{L} < \frac{1}{2}$	deep water $\frac{d}{L} > \frac{1}{2}$
1. wave profile	same as in transitional water	$\eta = \frac{H}{2} cos\left[\frac{2\pi x}{L} - \frac{2\pi t}{T}\right] = \frac{H}{2} cos\,\theta$	same as in transitional water
2. wave celerity	$C = \frac{L}{T} = \sqrt{gd}$	$C = \frac{L}{T} = \frac{gT}{2\pi} tanh\left(\frac{2\pi d}{L}\right)$	$C = C_0 = \frac{L}{T} = \frac{gT}{2\pi}$
3. wave length	$L = T\sqrt{gd} = CT$	$L = \frac{gT^2}{2\pi} tanh\left(\frac{2\pi d}{L}\right)$	$L = L_0 = \frac{gT^2}{2\pi} = C_0 T$
4. group velocity	$C_g = C = \sqrt{gd}$	$C_g = nC = \frac{1}{2}\left[1 + \frac{4\pi d / L}{sinh(4\pi d / L)}\right] \cdot C$	$C_g = \frac{1}{2} C = \frac{gT}{4\pi}$
5. water particle velocity a) horizontal	$u = \frac{H}{2}\sqrt{\frac{g}{d}} cos\,\theta$	$u = \omega \frac{H}{2} \frac{cosh[2\pi(z+d)/L]}{sinh(2\pi d/L)} cos\,\theta$	$u = \frac{\pi H}{T} e^{\frac{2\pi z}{L}} cos\,\theta$
b) vertical	$w = \frac{H\pi}{T}\left(1 + \frac{z}{d}\right) sin\,\theta$	$w = \omega \frac{H}{2} \frac{sinh[2\pi(z+d)/L]}{sinh(2\pi d/L)} sin\,\theta$	$w = \frac{\pi H}{T} e^{\frac{2\pi z}{L}} sin\,\theta$
6. water particle accelerations a) horizontal	$a_x = \frac{H\pi}{T}\sqrt{\frac{g}{d}} sin\,\theta$	$a_x = \frac{g\pi H}{L} \frac{cosh[2\pi(z+d)/L]}{cosh(2\pi d/L)} sin\,\theta$	$a_x = 2H\left(\frac{\pi}{T}\right)^2 e^{\frac{2\pi z}{L}} sin\,\theta$
b) vertical	$a_z = -2H\left(\frac{\pi}{T}\right)^2 \left(1 + \frac{z}{d}\right) cos\,\theta$	$a_z = \frac{g\pi H}{L} \frac{sinh[2\pi(z+d)/L]}{cosh(2\pi d/L)} cos\,\theta$	$a_z = -2H\left(\frac{\pi}{T}\right)^2 e^{\frac{2\pi z}{L}} cos\,\theta$
7. water particle displacements a) horizontal	$\xi = -\frac{HT}{4\pi}\sqrt{\frac{g}{d}} sin\,\theta$	$\xi = -\frac{H}{2} \frac{cosh[2\pi(z+d)/L]}{sinh(2\pi d/L)} sin\,\theta$	$\xi = -\frac{H}{2} e^{\frac{2\pi z}{L}} sin\,\theta$
b) vertical	$\zeta = \frac{H}{2}\left(1 + \frac{z}{d}\right) cos\,\theta$	$\zeta = \frac{H}{2} \frac{sinh[2\pi(z+d)/L]}{sinh(2\pi d/L)} cos\,\theta$	$\zeta = \frac{H}{2} e^{\frac{2\pi z}{L}} cos\,\theta$
8. subsurface pressure	$p = \rho\, g(\eta - z)$	$p = \rho g \eta \frac{cosh[2\pi(z+d)/L]}{cosh(2\pi d/L)} - \rho g$	$p = \rho g \eta\, e^{\frac{2\pi z}{L}} - \rho g z$

Table 4-2 Linear wave theory - wave characteristics

Orbital Motion

Still, within the linear small amplitude wave theory it is possible to derive the periodic changes of the position, the velocity and the acceleration of water particles. In deep water, the particles follow a circular pattern, in shallower water the circular motion is transformed into an elliptical pattern. This orbital motion can be described by splitting position, velocity and acceleration into horizontal and vertical components (ξ and ζ, u and w, and a_x and a_z respectively).

$$\xi = -\frac{H}{2}\frac{\cosh\left[2\pi(z+h)/L\right]}{\sinh(2\pi h/L)}\sin\theta \qquad \zeta = \frac{H}{2}\frac{\sinh\left[2\pi(z+h)/L\right]}{\sinh(2\pi h/L)}\cos\theta \qquad (4.20)$$

$$u = \omega\frac{H}{2}\frac{\cosh\left[2\pi(z+h)/L\right]}{\sinh(2\pi h/L)}\cos\theta \qquad w = \omega\frac{H}{2}\frac{\sinh\left[2\pi(z+h)/L\right]}{\sinh(2\pi h/L)}\sin\theta \qquad (4.21)$$

Note that at the water surface (z = 0), the expressions for ζ and η become identical, and that at the bottom, for z = $-h$, all vertical movements become zero due the influence of sinh (0).

Finally, there is a similar expression for the pressure at a level z below the water surface:

$$p = -\rho g z + \rho g \eta \frac{\cosh\left[2\pi(z+h)/L\right]}{\cosh(2\pi h/L)} \tag{4.22}$$

In this expression $\rho g z$ represents the hydrostatic pressure, and $\rho g \eta$ cosh(..) / cosh(..) the harmonic component of the pressure.

Wave Energy and Group Speed

If we examine a finite number of waves (group) in otherwise still water, we will observe that waves seem to originate at the rear of the group, move forward through the group and die out near the front of the group. Since the celerity of the individual wave was called c, we must conclude that there is a group velocity c_g, which is smaller than c. Linear theory shows that:

$$c_g = \frac{c}{2}\left(1 + \frac{2kh}{\sinh(2kh)}\right) \tag{4.23}$$

The ratio between group velocity and celerity c/c_g is often symbolised by the letter n.

The energy contained in a wave of unit width (1 m), and length L is:

$$E_T = \frac{1}{8}\rho g H^2 L \tag{4.24}$$

Often it is more convenient to express the energy in terms of energy per square meter of water surface:

$$E = \frac{1}{8}\rho g H^2 \tag{4.25}$$

This energy is propagated at the wave group speed c_g, thus causing an energy flux U, with

$$U = E \cdot c_g = E \cdot n \cdot c \tag{4.26}$$

Waves entering shallow water

When a wave is approaching perpendicular to the coast, the depth decreases, but the energy flux must remain constant. It is further assumed that the wave period also remains constant. If the deep water condition is denoted by the subscript $_0$, and the condition at the limited water depth by subscript $_1$, then:

$$\frac{1}{8}\rho g H_1^2 n_1 c_1 = \frac{1}{8}\rho g H_0^2 n_0 c_0 \tag{4.27}$$

since $n_0 = ½$, this expression can be rewritten as:

$$\frac{H_1}{H_0} = \sqrt{\frac{c_0}{c_1}\frac{1}{2n_1}} = k_{sh} \tag{4.28}$$

or

$$\frac{H_1}{H_0} = \sqrt{\frac{1}{\tanh(2\pi h/L)} + \frac{1}{1+\dfrac{(4\pi h/L)}{\sinh(4\pi h/L)}}} = k_{sh} \tag{4.29}$$

The coefficient k_{sh}, or the shoaling coefficient indicates the change in wave height due to a variation in the water depth. Since the derivation is based on conservation of energy, this shoaling coefficient can only be used if there is no dissipation of energy. Note that the theory of shoaling is equally valid for decreasing as for increasing water depth. This means that a wave that passes over a local shoal resumes its original height if no breaking has occurred.

The result is shown in Figure 4-17, which clearly indicates that first wave heights are slightly reduced, before they increase considerably when the waves come nearer to the shore.

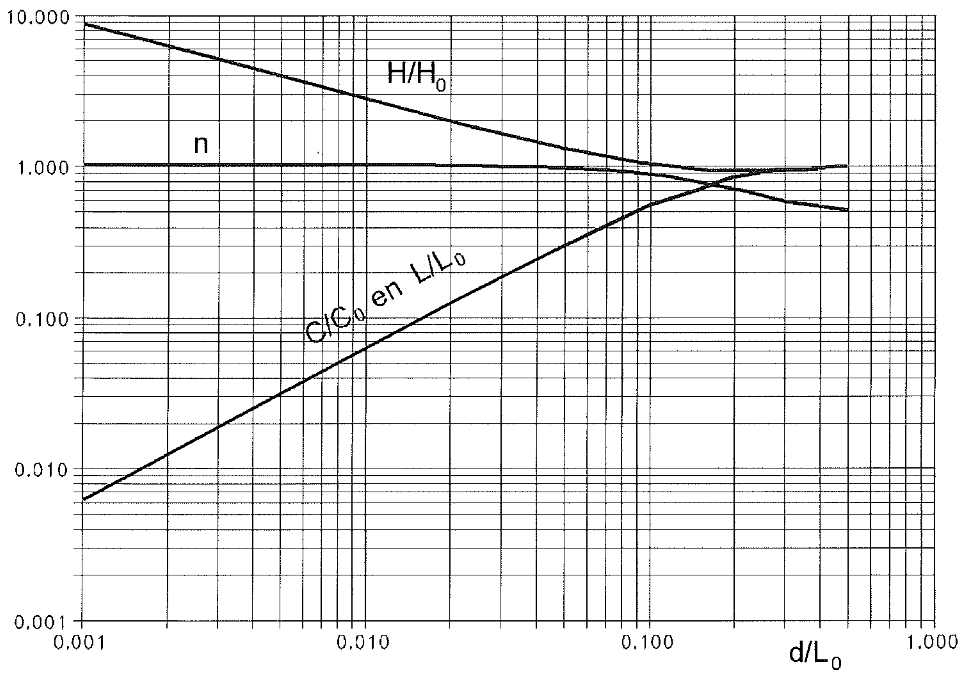


Figure 4-17 The effect of shoaling

This occurs only when no energy is dissipated, or in other words when no breaking takes place. Breaking and the limits of breaking are discussed a little later in this chapter.

When, due to increasing wave height, the conditions for the linear (small amplitude) wave theory are no longer fulfilled, deviations will occur. This means that different (often higher-order) theories have to be applied, taking into account deformation and the breaking of waves. A comprehensive mathematical review is presented by Miche (1944). A review of the validity of various wave theories is given by Le Mehaute (1969). See also Figure 4-18.

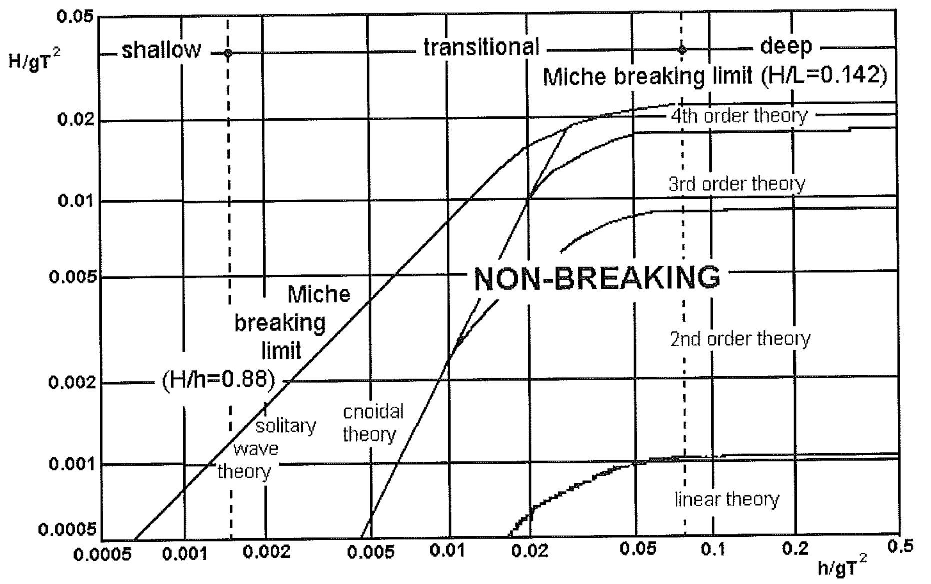


Figure 4-18 Validity of waves theories

One of the consequences of waves breaking in open water is that the crest of the wave becomes shorter and higher, and that the trough becomes less pronounced and longer. Due to this asymmetry, the orbital velocities in the direction of wave movement become higher, and the orbital velocities against the wave direction become smaller. However, it is remarkable how well the linear wave theory works, even beyond the strict limits of its validity!

The theory fails completely when waves approach the stage of breaking, either due to high steepness (H/L) or to entering shallow water (H/h). Theoretical limits are $H/L < 0.14$ and $H/h < 0.78$. These limits occur when the particle velocity in the crest exceeds the wave celerity, in other words when the water particles tend to leave the wave profile. This initiates a process of energy dissipation that considerably reduces the wave height.

Calculations of the shoaling coefficient are greatly facilitated by the availability of modern computers and calculators. Because an iteration process is required in some cases, it may be helpful to use standard tables containing the values of the hyperbolic functions. These tables can be found in the Shore Protection Manual. A brief extract of these tables is given in Table 4-3.

h/L_0	tanh(kh)	h/L	kh	sinh(kh)	cosh(kh)	H/H_0'	h/L_0	tanh(kh)	h/L	kh	sinh(kh)	cosh(kh)	H/H_0'
0.000	0.000	0.0000	0.000	0.000	1.000	∞	0.200	0.888	0.225	1.41	1.926	2.170	0.918
0.002	0.112	0.0179	0.112	0.112	1.006	2.12	0.210	0.899	0.234	1.47	2.060	2.290	0.920
0.004	0.158	0.0253	0.159	0.160	1.013	1.79	0.220	0.909	0.242	1.52	2.177	2.395	0.923
0.006	0.193	0.0311	0.195	0.196	1.019	1.62	0.230	0.918	0.251	1.57	2.299	2.507	0.926
0.008	0.222	0.0360	0.226	0.228	1.026	1.51	0.240	0.926	0.259	1.63	2.454	2.650	0.929
0.010	0.248	0.0403	0.253	0.256	1.032	1.43	0.250	0.933	0.268	1.68	2.590	2.776	0.932
0.015	0.302	0.0496	0.312	0.317	1.049	1.31	0.260	0.940	0.277	1.74	2.761	2.936	0.936
0.020	0.347	0.0576	0.362	0.370	1.066	1.23	0.270	0.946	0.285	1.79	2.911	3.078	0.939
0.025	0.386	0.0648	0.407	0.418	1.084	1.17	0.280	0.952	0.294	1.85	3.101	3.259	0.942
0.030	0.420	0.0713	0.448	0.463	1.102	1.13	0.290	0.957	0.303	1.90	3.268	3.418	0.946
0.035	0.452	0.0775	0.487	0.506	1.121	1.09	0.300	0.961	0.312	1.96	3.479	3.620	0.949
0.040	0.480	0.0833	0.523	0.547	1.140	1.06	0.310	0.965	0.321	2.02	3.703	3.835	0.952
0.045	0.507	0.0888	0.558	0.587	1.160	1.04	0.320	0.969	0.330	2.08	3.940	4.065	0.955
0.050	0.531	0.0942	0.592	0.627	1.180	1.02	0.330	0.972	0.339	2.13	4.148	4.267	0.958
0.055	0.554	0.0993	0.624	0.665	1.201	1.01	0.340	0.975	0.349	2.19	4.412	4.524	0.961
0.060	0.575	0.104	0.655	0.703	1.222	0.993	0.350	0.978	0.358	2.25	4.691	4.797	0.964
0.065	0.595	0.109	0.686	0.741	1.245	0.981	0.360	0.980	0.367	2.31	4.988	5.087	0.967
0.070	0.614	0.114	0.716	0.779	1.267	0.971	0.370	0.983	0.377	2.37	5.302	5.395	0.969
0.075	0.632	0.119	0.745	0.816	1.291	0.962	0.380	0.984	0.386	2.43	5.635	5.723	0.972
0.080	0.649	0.123	0.774	0.854	1.315	0.955	0.390	0.986	0.395	2.48	5.929	6.013	0.974
0.085	0.665	0.128	0.803	0.892	1.340	0.948	0.400	0.988	0.405	2.54	6.300	6.379	0.976
0.090	0.681	0.132	0.831	0.930	1.366	0.942	0.410	0.989	0.415	2.60	6.695	6.769	0.978
0.095	0.695	0.137	0.858	0.967	1.391	0.937	0.420	0.990	0.424	2.66	7.113	7.183	0.980
0.100	0.709	0.141	0.886	1.007	1.419	0.933	0.430	0.991	0.434	2.73	7.634	7.699	0.982
0.110	0.735	0.150	0.940	1.085	1.475	0.926	0.440	0.992	0.443	2.79	8.110	8.171	0.983
0.120	0.759	0.158	0.994	1.166	1.536	0.920	0.450	0.993	0.453	2.85	8.615	8.673	0.985
0.130	0.780	0.167	1.05	1.254	1.604	0.917	0.460	0.994	0.463	2.91	9.151	9.206	0.968
0.140	0.800	0.175	1.10	1.336	1.669	0.915	0.470	0.995	0.472	2.97	9.720	9.772	0.987
0.150	0.818	0.183	1.15	1.421	1.737	0.913	0.480	0.995	0.482	3.03	10.324	10.373	0.988
0.160	0.835	0.192	1.20	1.509	1.811	0.913	0.490	0.996	0.492	3.09	10.966	11.011	0.990
0.170	0.850	0.200	1.26	1.621	1.905	0.913	0.500	0.996	0.502	3.15	11.647	11.689	0.990
0.180	0.864	0.208	1.31	1.718	1.988	0.914	1.000	1.000	1.000	6.28	266.893	266.895	1.000
0.190	0.877	0.217	1.36	1.820	2.076	0.916	∞	1.000	∞	∞	∞	∞	1.000
0.200	0.888	0.225	1.41	1.926	2.170	0.918							

Table 4-3 Sinusoidal wave functions

Refraction

When waves travel from deep water into shallower water, some significant changes occur. From equations (4.15) and (4.19), it can clearly be seen that the wave celerity decreases with depth. When a wave approaches underwater contours at an angle, it is evident that the sections of the crest in the deeper parts travel faster than those in the shallower sectors. This causes the wave crest to turn towards the depth contour. This bending effect is called refraction, and is analogous to similar phenomena in physics (light, sound). The effect is shown in Figure 4-19.

The refraction theory, assumes that no wave energy moves laterally along the wave crest. The energy remains constant between orthogonals, normal to the wave crest. The direction of the orthogonals changes proportionally to the wave celerity according to the law of Snellius:

$$\sin\alpha_2 = \left(\frac{c_2}{c_1}\right)\sin\alpha_1 \tag{4.30}$$

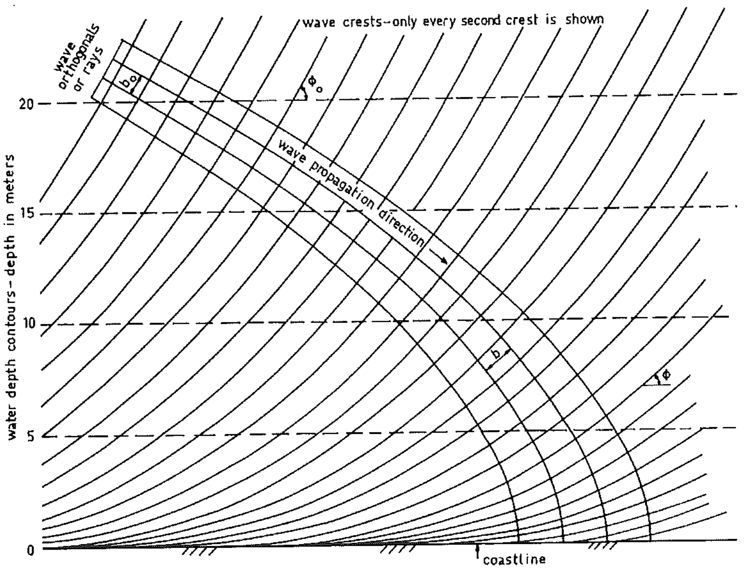


Figure 4-19 Wave refraction

By applying equation (4.30), it is possible to construct a field of orthogonals over a given bottom configuration for a given wave direction and wave period. During this process, the distance b between the orthogonals may vary. Considerations of energy conservation then show that:

$$\frac{H_2}{H_1} = \sqrt{\frac{b_1}{b_2}} \tag{4.31}$$

The factor $\sqrt{(b_1 / b_2)}$ is also called the refraction factor k_r and is used to calculate the change in wave height when a wave approaches at an angle to the shore. Designing a field of orthogonals is a cumbersome task, specifically when it has to be repeated for various wave directions and various wave periods. The outcome is not always satisfactory; and certainly not when, owing to the bottom topography some of the orthogonals intersect. This leads to infinitely high wave heights!

Nowadays, mathematical models are available to calculate the transport of wave energy through a grid. The effects of shoaling, refraction, bottom friction and wind can be incorporated in the models. Examples of such models are HISWA and SWAN, both developed at Delft University of Technology. Use of these models eliminates need to design of wave rays. Estimating (not calculating) wave rays is still a quick method by means of which qualitative (not quantitative) answers can be obtained.

Diffraction

When we discussed refraction, we assumed that no lateral transfer of wave energy would take place along the wave crest. This assumption is correct as long as the lateral gradient is not too great. However, this assumption is no longer valid when an infinitely high gradient occurs, for instance when at one location the wave energy is allowed to pass, whereas next to it, the wave propagation is prevented by an obstruction (island or breakwater). In such case there is some lateral transfer of wave energy. The phenomenon can clearly be distinguished in Figure 4-20 Typical Example of Diffraction.

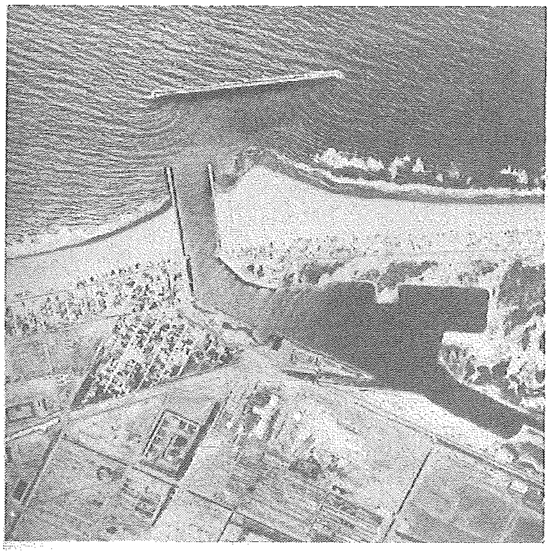


Figure 4-20 Diffraction pattern

The theory of wave diffraction is solved mathematically by application of the "Cornu-spiral". Practical contours for considerations about diffraction are given in the Shore Protection Manual, Anonymous (1984).

4.7.2. Breaking

When waves approach the shore, the wave celerity is reduced. In addition, we have seen that the wave height increases due to shoaling and the orbital motion within the waves is also changing. Although the orbit in deep water is a circle, in shallow water the orbit becomes an ellipse with a horizontal axis longer than the vertical axis. The vertical axis of the orbit at the water surface is equal to the wave height. Since shoaling causes an increase in the wave height, the vertical motion of the water particles at the surface must also increase. In addition to this, the horizontal movements grow in relation to the vertical movements, which means that there must be a significant increase of the particle velocity near the surface. When the particle velocity exceeds the wave celerity, the crest of the wave is no longer stable, so breaking must occur. Since most wave theories are based on energy conservation, their validity ends when energy dissipation takes place. The assumption that the wave period remains constant is also no longer true. This has very serious consequences on the design of coastal structures, specifically when the wave period is part of the design formulae.

The process of breaking takes place in various different ways. We can recognise distinct types of breakers. A distinction is made between spilling, plunging and surging breakers (Figure 4-21). The parameter that guides the breaker type is (Battjes [1974]):

$$\xi = \frac{\tan\alpha}{\sqrt{H/L_0}} \tag{4.32}$$

where:
α = steepness of the beach
L_0 = wave length in deep water

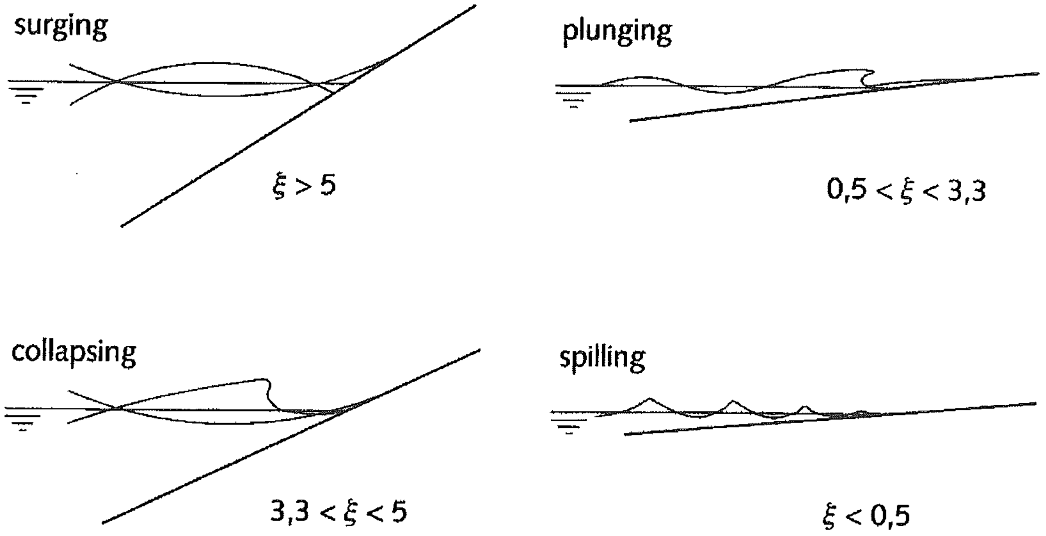


Figure 4-21 Breaker types

Spilling breakers are usually found along flat beaches. Waves begin breaking at a relatively great distance from shore and break gradually as they approach still shallower water. During breaking, a foam line develops at the crest and leaves a thin layer of foam over a considerable distance. There is very little reflection of wave energy back towards the sea.

A plunging breaker is a type that is often found on the travel posters for the Pacific Islands with a beautiful windsurf professional below it; it is spectacular. The curling top is characteristic of such a wave. When it breaks, a lot of energy is dissipated into turbulence; little is reflected back to the sea, and a little is transmitted towards the coast, while forming a "new" wave.

Surging breakers occur along rather steep shores such as might be encountered along rocky coasts. The breaker zone is very narrow, and more than half of the energy is reflected back into deeper water. The breakers form up like plunging breakers, but the toe of each wave surges upon the beach before the crest can curl over and fall.

4.7.3 Irregular waves

Unlike what is stated in the wave theory discussed in the previous sections, natural waves are neither small amplitude waves, nor regular in character with respect of Height H and period T.

Irregularity occurs on at least two distinctly different time scales, characterised by the short term and the long tern variations respectively. The easiest way to distinguish these two phenomena is to assume that during a particular storm, the wave pattern is stationary. In other words, we neglect the gradual growth and decay of the wave field, and we consider the storm more or less as a block function. Even then, the wave motion is irregular, as is demonstrated by the wave record shown in Figure 4-22.

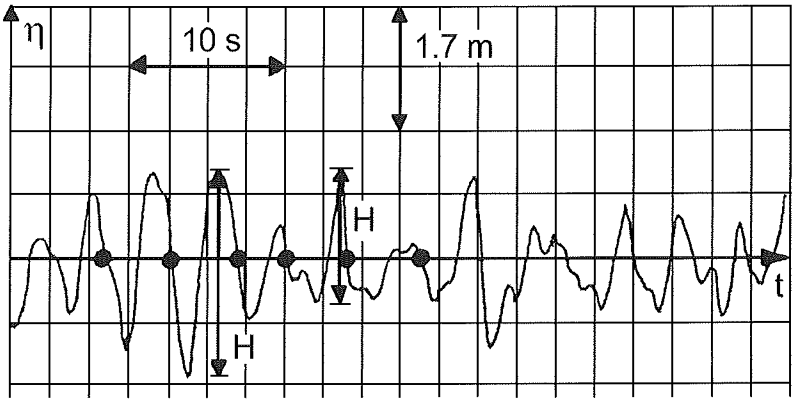


Figure 4-22 Irregular wave

Short Term Statistics
Individual waves can be distinguished according to international standards by considering the water surface elevation between two subsequent upward or downward crossings of the mean water level. The time span between these crossings is the wave period; the range between the highest crest and the lowest trough is the wave height. Since all heights and periods of individual waves are different, it is logical to apply statistical methods to characterise the set of data. The easiest way is to determine the statistical properties of the wave heights only.

It appears that in deep water, the probability of exceedance of wave heights follows a Rayleigh distribution:

$$P(\underline{H} > H) = e^{-2\left(\frac{H}{H_s}\right)^2} \tag{4.33}$$

In which H_s, the significant wave height is equal to the average of the 1/3 highest waves. H can also be defined as the wave height that is exceeded by 13.5% of the waves. A third definition and method to determine H_s is given later.

A graphical presentation of the recorded wave pattern is often made on so-called Rayleigh graph paper (Figure 4-23). On such paper, a data set that follows the Rayleigh distribution is represented by a straight line through the origin. In view of the definitions, the value of H_s can be read at the 13.5% exceedance value. In this way, the strength of the storm considered is apparently determined by just one value: H_s. A stronger storm would lead to a steeper distribution curve, which is again defined by a specific value of the significant wave height.

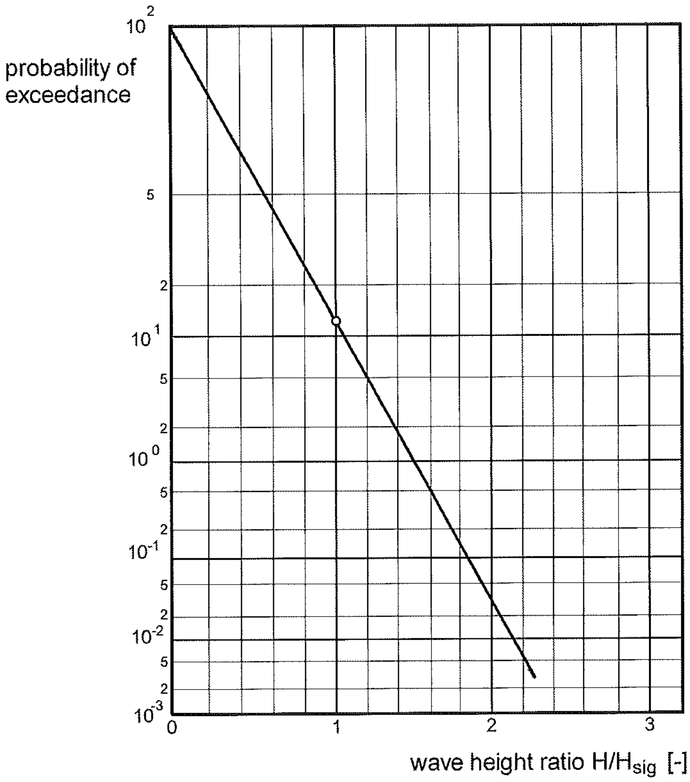


Figure 4-23 "Rayleigh – paper"

Wave periods are generally treated in a slightly different way. It is possible to consider the irregular surface level $\eta(t)$ to be the sum of a large number of periodic waves:

$$\eta(t) = \sum a_i \cos(2\pi f_i t + \varphi_i) \tag{4.34}$$

In which
a_i = amplitude of component i
f_i = $1/T_i$ = frequency of component i
φ_i = phase angle of component i

The spectral energy density $S(\omega)$ can then be expressed as:

$$S(\omega) = 1/2 \sum^{\Delta\omega} a_i^2 / \Delta\omega \tag{4.35}$$

The energy contained in the entire frequency range is proportional to $\overline{\eta^2}$, and denoted by m_0. Consequently:

$$m_0 = \overline{\eta^2} = \tfrac{1}{2} \sum_{i=1}^{n} a_i^2 \tag{4.36}$$

In practice the determination of $S(\omega)$ based on a more mathematical concept using the autocorrelation function $R(\tau)$ and its Fourier transform. In this process, some mathematical hiccups may occur. It is therefore recommended to ascertain that a direct analysis of the wave height distribution yields the same significant wave height as the spectral approach. In other words, check whether

$$H_s = 4\sqrt{m_0} = H_{13.5\%} \tag{4.37}$$

When the wave energy spectrum has been established in this way, in most cases it is possible to distinguish a frequency $f = 1/T$ or period T where the maximum energy is concentrated. This value is called the peak period T_p. Of course, one can also count the total number of waves during the recording period and thus define an average period.

It is stressed that the spectral analysis and the Rayleigh distribution are only valid to analyse a stationary process. It is necessary to ensure that the chosen measuring period that is not so long that it is almost certain that the wave climate will change during the observation period. On the other hand, the chosen observation period must be long enough to ensure that the sample leads to statistically reliable results. It has become common practice to measure waves during a period of 20 to 30 minutes at intervals of 3 or 6 hours.

Summarising, one can state that the short-term distribution of wave heights, i.e. the wave heights in a stationary sea state exhibits some very characteristic relations:

Name	**Notation**	$H/\sqrt{m_0}$	H / H_s
Standard deviation free surface	$\sigma_\eta = \sqrt{m_0}$	1	0.250
RMS height	H_{rms}	$2\sqrt{2}$	0.706
Mean Height	$\overline{H} = H_1$	$2\sqrt{\ln 2}$	0.588
Significant Height	$H_s = H_{1/3}$	4.005	1
Average of 1/10 highest waves	$H_{1/10}$	5.091	1.271
Average of 1/100 highest waves	$H_{1/100}$	6.672	1.666
Wave height exceeded by 2%	$H_{2\%}$		1.4

Table 4-4 Characteristic wave heights

Note: these relations are valid only for deep water, i.e. in the absence of breaking waves.

In a similar way, wave periods can be related:

Name	**Notation**	**Relation to spectral moment**	T / T_p
Peak period	T_p	$1 / f_p$	1
Mean period	T_m	$\sqrt{(m_0/m_2)}$	0.75 to 0.85
Significant period	T_s		0.9 to 0.95

Table 4-5 Characteristic wave periods

When irregular waves enter shallow water, the highest waves will break first. This means that the Rayleigh distribution is no longer applicable. The breaking limit for regular waves is H / h=0.78, but for irregular waves the value H / H_s=0.5 is often used as the breaking limit. This indicates that for breaking waves H_s will not increase beyond 50% of the water depth.

Long Term Statistics

As indicated above there is no point in determining the either the significant wave height or a spectrum if the wave train is not part of a stationary process. Therefore, waves are measured at regular intervals of 3 to 6 hours during a relatively short period. It is highly unlikely that two subsequent observations will lead to identical values of H_s and T_p.

The results of series of wave observations covering a longer period will therefore again become a set of random data that represent the long-term wave climate of the location.

For some problems, it is sufficient to express this long-term wave climate in terms of the probability of exceedance of storms with a particular strength (H_s) but for others, it is necessary to have an idea of the probabilities of individual wave heights occurring. In such cases, the short term and the long-term expectations must be combined into a graph or table that expresses the probability of exceedance of individual wave heights during a fixed period, for example the lifetime of a structure.

Because in shallow water there is a direct relation between maximum breaking wave height and water depth, the wave height distribution is not independent of the occurrence of extreme water levels. Close to the shore, this could mean that the long-term distribution of wave heights coincides with the distribution of extreme water levels as given in section 4.4.4.

5. COASTAL MORPHOLOGY

5.1 Introduction

In the coastal zone, oceanography, geology, ecology and morphology are strongly interrelated. There are three types of processes that influence the configuration of the coast: physical, chemical, and biological processes. Foremost are the physical processes: tides, waves, winds and currents that continuously influence the main coastal features created during the geological history. These processes wear down the coast in some places and build it up in others. Transport of all kind of sediments (sand, clay and shell) plays an important role in these processes. Barriers, spits, the shape of a coastal bay, and the course of a river, are all features dominated by sediment transport. Transgression and progradation of coasts are the result of both, sediment transport and other, less visible geological events.

Morphology is best understood when it is considered as a sediment balance for a given situation and a given balance area. In such a balance, all processes with sediment-transporting capacities must be taken into account. In this chapter, these processes are presented with emphasis on coastal morphology. Many of them take place in the surf zone; therefore the surf zone is described separately in section 5.2. The sediment transporting mechanisms are treated in section 5.3. Section 5.4 is focussed on coastline changes and coastline equilibrium in general. In section 5.5, attention is paid to the quantification of the transport processes, which makes them accessible for an engineering approach. Eventually, the morphological processes, together with certain geological features, lead to clearly different coastal formations, which are discussed in Chapter 6.

Morphological processes are strongly correlated. Sand transport is caused by waves, wind and currents, but waves, wind and currents also influence each other. Moreover, the sand transport causes changes in the topography, which in turn lead to changes in the wave wind and current patterns. In Figure 5-1, a scheme of this coherence in the morphological system is shown. It is a complex system. The elements (input variables) are:

- original coastal topography
- water level
- wind
- waves
- tide

The processes driven by the input variables lead to sand transport, which eventually leads to a coastal topography that changes as a function of time.

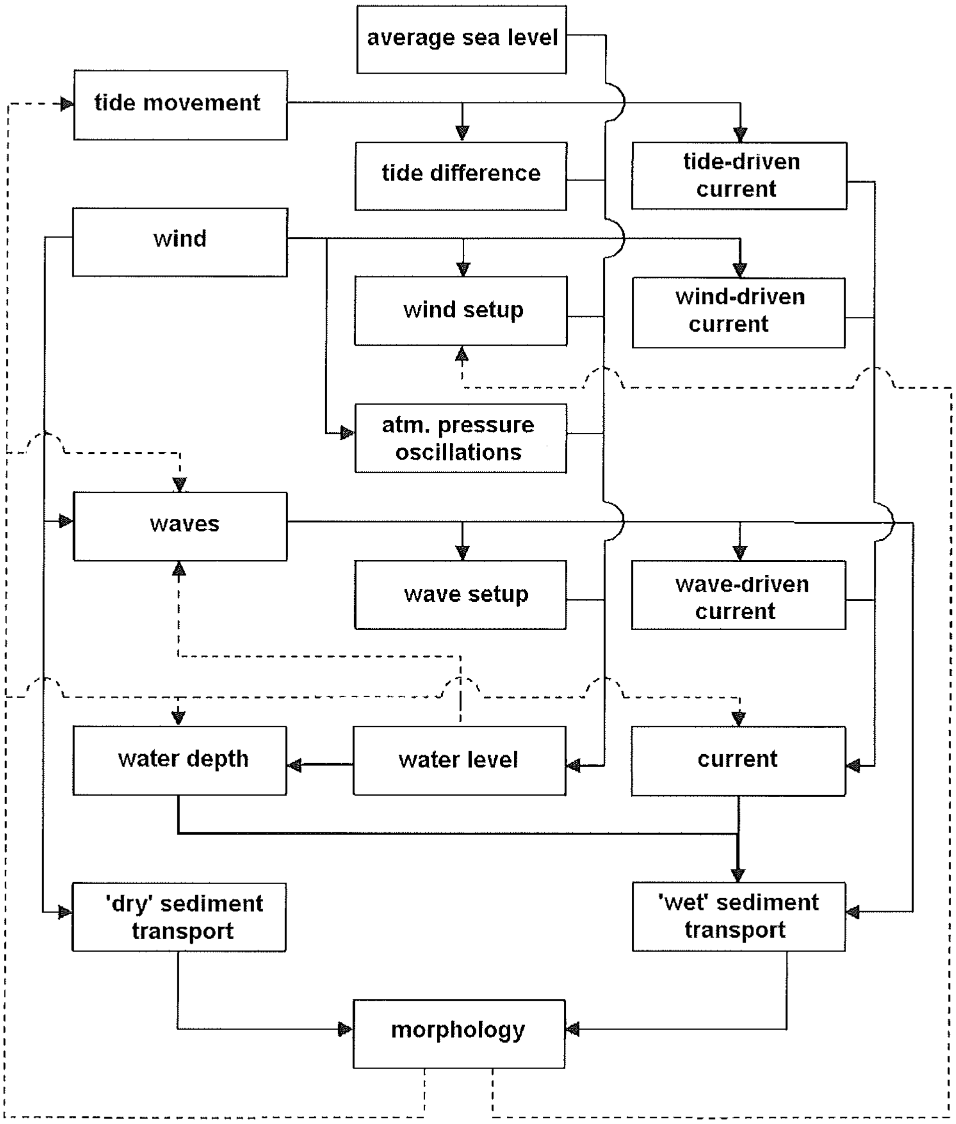


Figure 5-1 The morphological system

When studying morphological processes, we must realise that we can do so on completely different scale levels. For example, we may study what happens instantaneously in the boundary layer under a wave in a flume in the laboratory. The time scale is then in the order of seconds, the length scale in the order of metres. We may also study the erosion near the toe of a seawall. In that case the time scale will be months or maybe years, and the length scale will be hundreds of metres. When we consider the stability of an entire coastline, the time scale may be centuries and the length scale hundreds of kilometres. At first sight it will be difficult to integrate these approaches, but research on the small-scale is sometimes indispensable if one wishes to understand the large-scale effects. At the same time, we must remember that in the present state-of-the-art it is virtually impossible to predict the long-term effects from a simple integration of small-scale processes.

5.2 Surf zone processes

In the surf zone, complex hydrodynamic processes take place. The most obvious one is the energy dissipation due to breaking. It can easily be observed that wave energy is transformed into turbulence and noise. Less visible, but still very important, is the change of the mean water level in the surf zone. *Mathematically*, this can be derived from a calculation of the transport of momentum. This phenomenon is called radiation stress. For details, one is referred to a textbook on short waves (Battjes 1986). For waves approaching perpendicular to the shore this leads to a slight decrease of the mean water level outside the surf zone; inside the surf zone it leads to a rise in the mean level when one gets closer to the shore (wave setup). *Physically*, we can understand that in the crests of breaking waves an excess of water is transported towards the shore. This must lead to a rise in water level close to the shoreline. The gradient thus formed drives an undertow in seaward direction that compensates the mass transport in the crest of the breaker.

When the waves approach the shore at an angle, the process described above will also create a current parallel to the shore. Since the wave setup takes place in the breaker zone, this longshore current will also be concentrated in the breaker zone (See Figure 5-2)

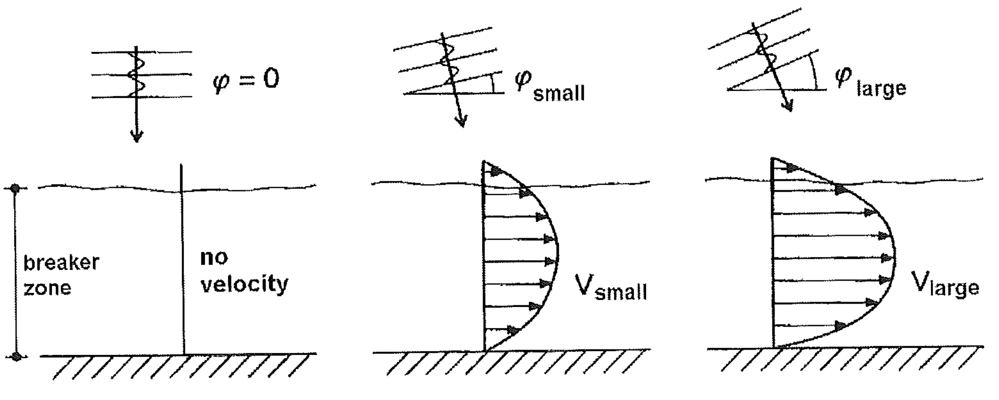


Figure 5-2 Longshore current

In a real situation, with slight irregularities, the undertow will not be evenly distributed along the length of the beach. The return flow will – at least partially- concentrate into so-called rip currents (in Dutch: muistroom). In this way, both vertical cells and horizontal cells can be distinguished. In Figure 5-3 and Figure 5-4, these different current patterns are shown.

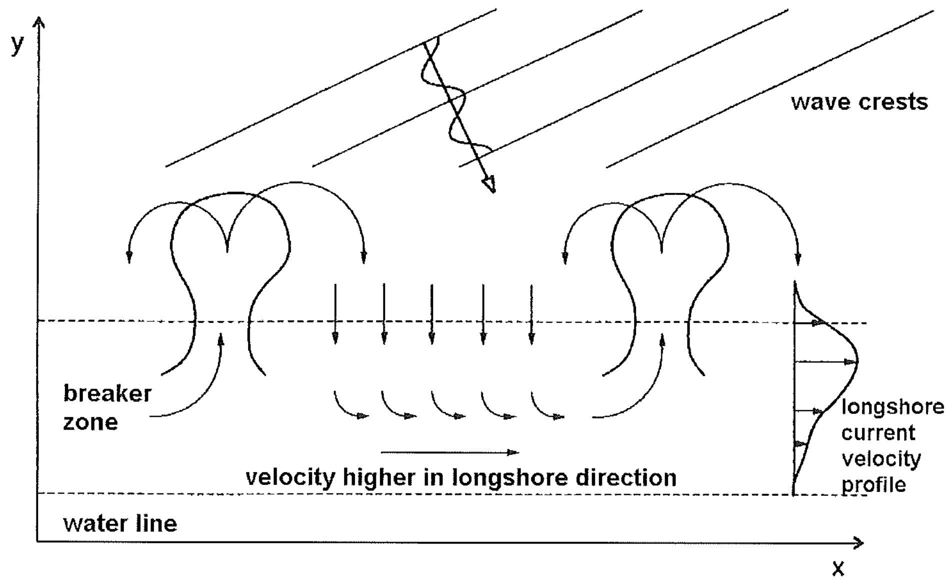


Figure 5-3 Horizontal circulation cell with rip current

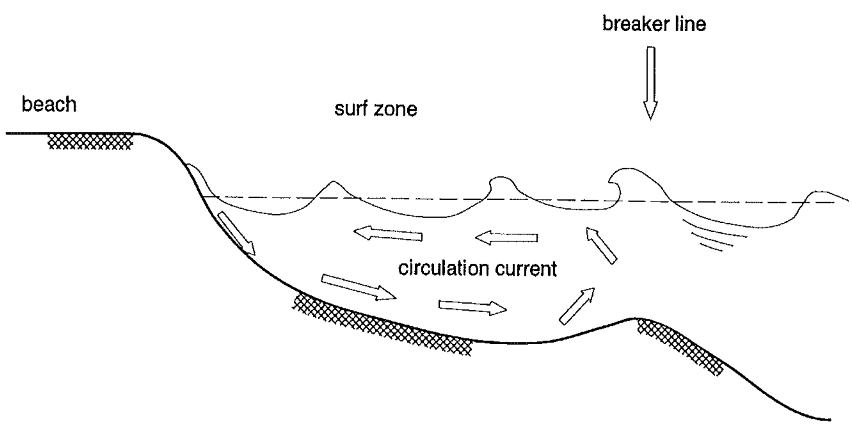


Figure 5-4 Vertical circulation cell with undertow

5.3 Sediment transport

Transport by waves and currents

Sediment transport plays an important role in nearly every coastal engineering problem. An important goal of coastal engineering is to understand the natural morphological changes due to sediment transport and to predict the sediment transport rates along a coast in general and in the vicinity of structures in particular. Frequently a shortage of material occurs at some location (undesired erosion); while at other places an overabundance of material can be just as troublesome (siltation of a navigation channel, for example).

When discussing sediment transport, it is common to distinguish bottom transport and transport in suspension. For sediment transport in rivers both modes of transport are sometimes combined into one general formula. In many cases this is a valid simplification because the direction of the transport is identical due to the unidirectional character of the flow. Along the coast, we have to be careful with such simplifications because bottom transport and suspended transport need not follow the same path. This may be demonstrated by the example of a wave that approaches the shore perpendicularly. It creates only orbital velocities normal to the coast. This means that bottom transport is also restricted to the cross-shore direction. Material that comes into suspension, however, for instance in the breaker zone under the influence of breaking waves, can be transported parallel to the coast under the influence of a weak tidal current along the shore, even if this current would be too weak to cause any bottom transport.

There are many formulae for the quantification of sediment transport. Most of these formulae are rather complicated and none of them provides an ideal answer to all questions. It is important to realise that it is common practice to accept a threshold value for bottom shear stress introduced by Shields. Apart from that, most transport formulae used in rivers and canals express the sediment transport per unit width s as:

$$s = mU^n \tag{5.2}$$

in which

s = sediment transport in $m^3/s/m$(width)
m = coefficient
U = (average) current velocity in m/s
n = power varying from 3 to 5

From the theory of sediment transport, we know that turbulence enhances the pick-up and transport of sediment, since it causes sediment to be in suspension. When suspended, the sediment will be carried by any resulting net current, the transport rate being the product of instantaneous velocity and concentration. We must therefore expect that the breaker zone is a very active zone where sediment transport is concerned. Interruption or weakening of the breaking process automatically leads to less turbulence and thus to less material in suspension. At the same time, it leads to a reduction of the wave driven longshore current.

Calculations in coastal engineering tend to be more difficult than similar predictions for river sediment transport; oscillating water movements caused by waves and the multitude of current-causing forces considerably increase the number of variables involved. Another complication is the non-stationary behaviour of both current velocity and sediment concentration.

Suspended sediment transport in a vertical can be described quite generally as the product of velocity u and sediment concentration c. Both are a function of time (t) and location (z) as indicated in equation (5.3).

$$S(t) = \int_{-h}^{o} c(z,t)\ u(z,t)\, dz \tag{5.3}$$

and:

$$S = \frac{1}{t_1} \int_{-h}^{o} \int_{0}^{t_1} c(z,t)\, u(z,t)\, dzdt \tag{5.4}$$

Solving equations (5.3) and (5.4) is not simple, certainly not when the sediment transport is at least partly caused by waves. For instance, Figure 5-5 shows the sediment concentration as a function of time for a large number of individual records.

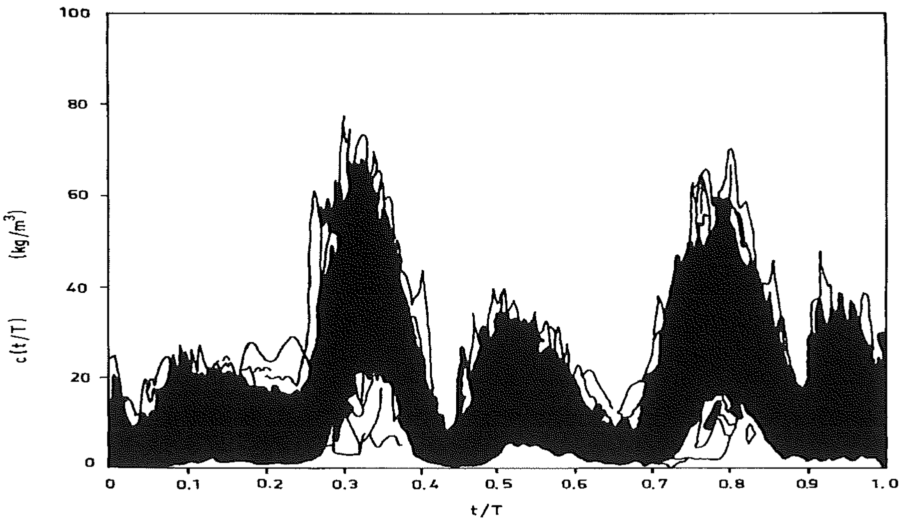


Figure 5-5 Sediment concentration as a function of time

If the sediment concentration c is constant in time, as is the case for a stationary current, equation (5.3) can be simplified into:

$$S(t) = S = \int_{-h}^{0} c(z) \cdot u(z) dz \tag{5.5}$$

Transport by wind

Along the coast, sediment transport by wind should not be neglected, although it is often overlooked. Sediment transport by wind plays a dominant role in the formation of dunes and thus in the shape of the coast immediately landward of the waterline.

The process of dispersive movement by wind is dependent on geometry and vegetation (Figure 5-6 and Table 5-1).

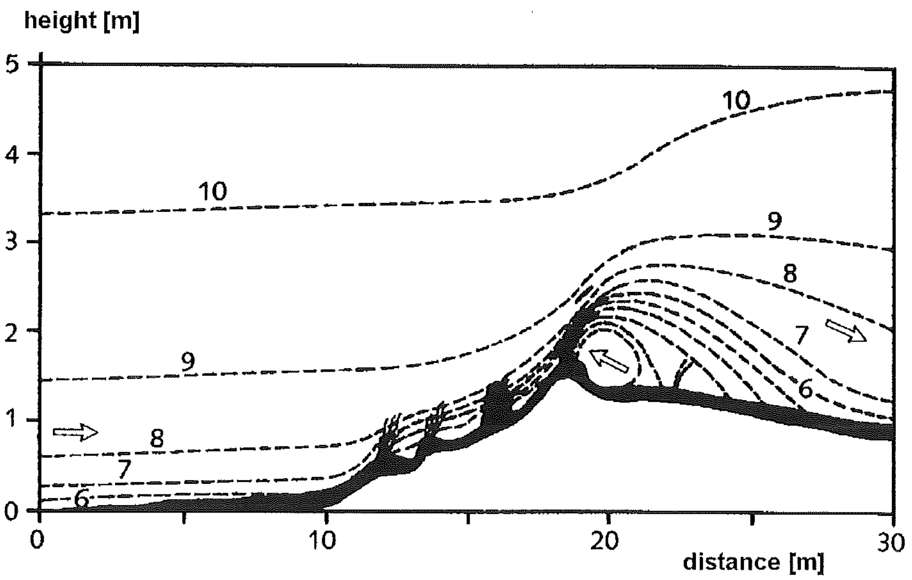


Figure 5-6 Wind blowing over a vegetated dune

wind force (Beaufort)	wind speed (m/s) (approx.)	sediment transport ($10^{-6}.m^3/s/m$)
3	4.5	-
4	7.0	1
5	10.0	3
6	12.5	14
7	15.5	31
8	19.5	86
9	22.5	165
10	26.5	310
11	31.0	408

Table 5-1 Correlation between wind force, wind velocity and blown sand transport

5.4 Coastline changes and coastline equilibrium

When considering sediment transport in the coastal zone, it is convenient to distinguish sediment transport normal to the coast (cross-shore transport) and sediment transport parallel to the coast (longshore transport) (Figure 5-7). Cross-shore transport and longshore transport are determined by the prevailing morphological and hydraulic conditions. Cross-shore transport is generally caused by the orbital velocities and, in the case of breaking waves, by the undertow. Gravity along the slope also plays a role. Longshore transport is usually caused by the longshore current that is driven by radiation stress of waves approaching under an angle. Cross-shore transport is often a combination of bottom transport and suspended transport, longshore transport is dominated by suspended transport.

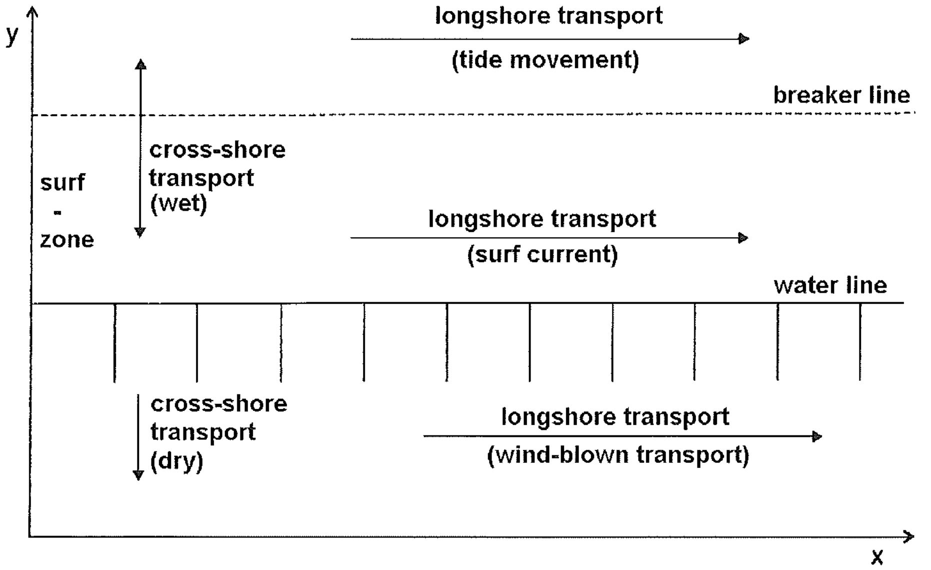


Figure 5-7 Longshore and cross shore transport

Sediment transport in itself does not cause any changes in the topography of the coastline. Only when there are gradients in the transport rate ($\partial s/\partial x$, $\partial s/\partial y$) will there be erosion or sedimentation. Gradients in the cross shore direction will lead to a steeper or more gentle slope, gradients in the longshore direction will lead to systematic erosion or sedimentation along the coastline.
In Chapter 4, we have already seen that moderate (non-breaking) waves in shoaling water show an asymmetry in the orbital velocity near the bottom. Velocities towards the shore are higher than

velocities in the opposite direction. Although the duration of the shoreward velocity is shorter, the power relation between sediment transport and velocity (see equation (5.4)) causes a net shoreward sediment transport.

Breaking waves, however, cause a net shoreward mass (water) transport, which results in a wave set-up shoreward of the breaker zone. This process ultimately leads to the creation of an undertow along the bottom towards the sea. Since a lot of sediment is in suspension in the breaker zone due to turbulence, the undertow can carry considerable amounts of sediment in a seaward direction.

The general shape of a coastal profile is the result of a dynamic equilibrium in the cross-shore direction. Such an equilibrium profile can be characterised by its slope. This slope depends on the wave height, the grain size and the distance from the shore. Higher waves and finer sand will cause a more gentle profile, whereas lower waves and coarser sand give rise to a steeper profile.

Based on a large number of field observations, Wiegel (1964) presented a graph indicating the relation between slope and grain size for various wave conditions (Figure 5-8).

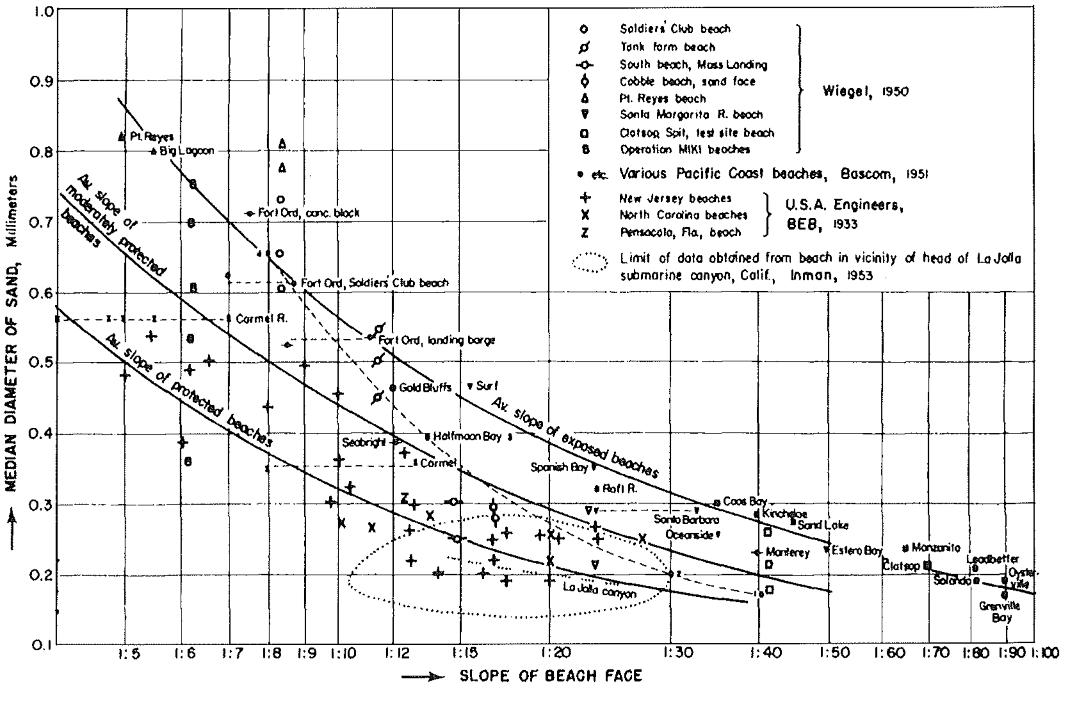


Figure 5-8 Beach slope versus grain size after Wiegel (1964)

Dean (1983) suggested a mathematical description of the equilibrium profile:

$$h = Ay^m \tag{5.6}$$

in which h is the water depth at a distance y from the shoreline. Various researchers have used and modified this formula, indicating values for A and m depending on grain size, wave climate, water level variation and tidal currents. The most common value for m is in the order of 2/3; the coefficient A is often replaced by $B.D^{1/3}$, indicating the influence of the grain size D.

Vellinga (1984) describes the slope that develops when a dune coast is exposed to a storm:

$$\left(\frac{7.6}{H_{so}}\right)y = 0.4714\left[\left(\frac{7.6}{H_{so}}\right)^{1.28}\left(\frac{w}{0.0268}\right)^{0.56}x + 18\right]^{0.5} - 2.0 \qquad (5.7)$$

In which:

H_{so} = significant wave height in deep water (m)
w = fall velocity of beach sand in sea water of 5°C (m/s)
x = horizontal distance to shoreline (= dune base)
y = depth below water level associated with equilibrium profile

For a better understanding refer to Figure 5-9.

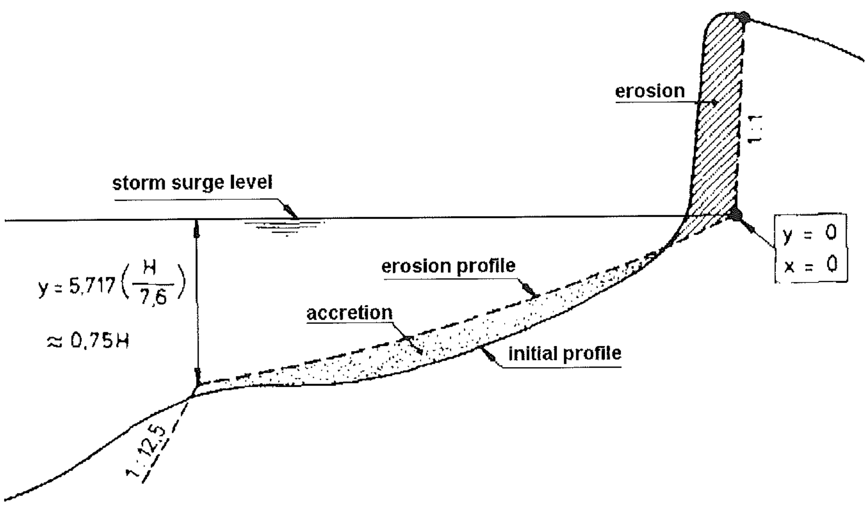


Figure 5-9 Beach profile after dune erosion

Equation (5.7) describes the curved profile from the waterline (x=0, y=0) to the point where the gently curving profile ends and the seabed has a more or less steep slope of 1:12.5. This point has coordinates:

$$x = 250\left(\frac{h}{7.6}\right)^{1.28}\left(0.0268/w\right)^{0.56}$$

and

$$y = 5.717\left(H/7.6\right) \approx 0.75H$$

In practice, the curved profile, as given by Dean, is sometimes replaced by a straight slope from the water line down to a horizontal level that is not considered to change.
Because wave heights change during the season, the resulting profiles are called summer and winter profiles. Extreme storms, specifically storms that are associated with a rise in mean sea level, may create such a gentle profile that the required sand for this profile is eroded from the dunes. If no sand is permanently lost from the cross section, the original condition is restored by nature in the long run.

In this way, during a short time (a storm), a change of the profile can occur that creates the impression of severe erosion. This is not the same as permanent, structural erosion, which is often due to a longshore transport gradient. NB: The difference between structural and temporary erosion cannot be seen above the water surface. For more details about coast profiles and erosion see Section 7.4.

Gradients in the (longshore) transport can result from incident waves under a different angle, wave height differences along the coast, bottom material changes and wind and wave driven currents (Figure 5-10).

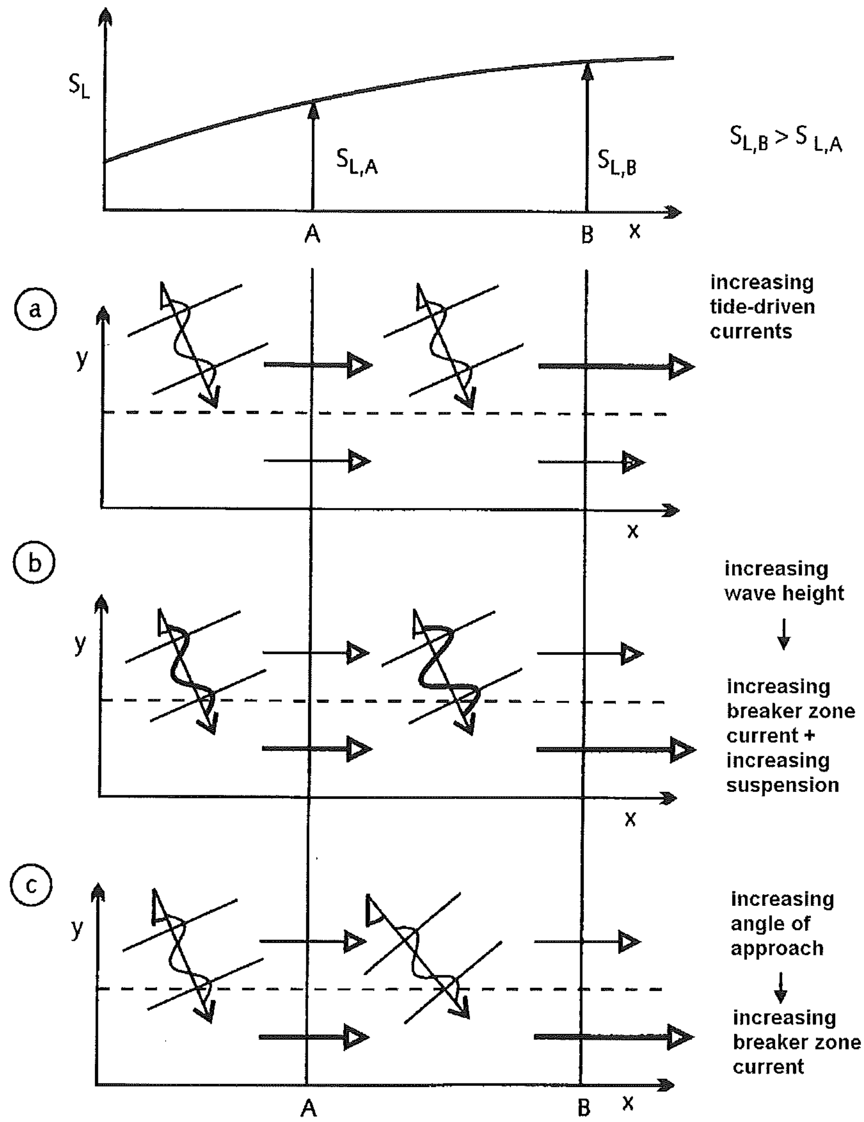


Figure 5-10 Causes of a gradient in longshore transport

5.5 Quantification of the longshore transport process

The longshore transport is essentially caused by waves approaching at an angle to the coastline. Due to wave breaking there is a zone with an increased turbulence level, where bottom material is brought into suspension. Once suspended, the material is transported by the longshore current that is caused by the breaking waves. The longshore transport is therefore concentrated in the breaker zone.

For a long time, the amount of sand transported in this way has been approximated by a formula developed by the Coastal Engineering Research Centre (CERC) of the US Army Corps of Engineers:

$$S = 0.020\, H_{s0}^2\, c_0\, K_r^2 \sin\phi_b \cos\phi_b \tag{5.8}$$

in which:

S = sand transport (m^3/s)
c_0 = wave celerity in deep water (m/s)
ϕ_{br} = angle between depth contours and wave crest at breaker line
H_{s0} = significant wave height at deep water (m)
K_r = refraction coefficient

The formula appears in different shapes, depending on the use of particular values and definitions for H, c, K and ϕ. Use of this formula is complicated because both wave height and wave direction vary throughout the year. For a reliable result, the wave climate over the year must be divided into a number of characteristic periods with conditions that are considered to be representative of certain periods of time. This leads to transport rates in two directions. The combination of these leads to a net transport rate. Net transport may vary from say 100,000 to 1,000,000 m^3 per annum.

Although there are more sophisticated expressions for the longshore transport rate, the final result is often not much better than the result of the CERC formula because of the great uncertainty with respect to the boundary conditions H and ϕ. It is remarkable that the CERC formula neglects any influence of grain size. It must also be remembered that the formula does not consider transport due to tidal currents if any.

Considering the formula in more detail, it is evident that longshore transport is zero when $\phi_0 = 90^\circ$, i.e. when the waves are approaching at right angles to the coast. The transport reaches a maximum for $\phi_b = 45^0$.

6. COASTAL FORMATIONS

6.1 Introduction

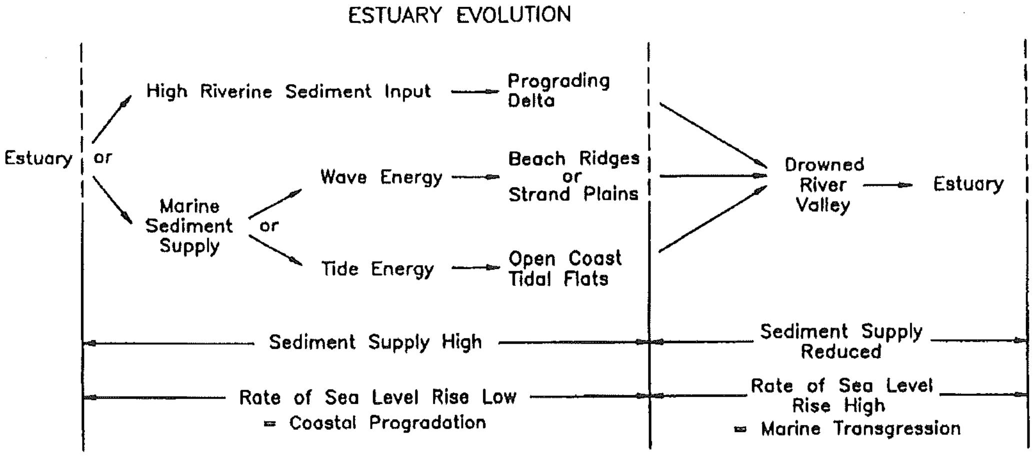
In the previous chapters, we have seen how coastlines have initially been formed by long term geological processes, and how climatic conditions, waves and tides are constantly changing the initial forms by erosion and the transportation of sediment. Although every location on earth has its own character caused by its history and by the prevailing conditions, general forms can still be recognised that give at least some preliminary insight into the processes that dominate the development of that specific location. In this chapter, an attempt is made to develop a systematic approach that leads to the identification of characteristic coastal shapes.

The central concept behind all attempts to understand coastal changes is the idea of two major steering processes: progradation and transgression. These processes shape a coast according to the sediment supply in relation to the relative sea-level rise. If the sea-level rise is high, and/or the sediment supply relatively low, then marine transgression of a coast is taking place. If the sea-level rise is low, in combination with a high sediment supply, then coastal progradation is happening. In Figure 6-1, this concept of prograding and transgressive coasts is shown.

The left side of Figure 6-1 represents prograding situations. Then the landside is on the winning hand, either because of a falling sea level relative to the land, or because of an excessive sediment supply. The right side represents the transgressive case, either because of a rise in sea level, or because of insufficient sediment supply. NB: the change in sea level is relative, meaning that subsidence of the land with a constant sea level has the same effect.

In the prograding case, deposition of river sediment leads to delta formation. When wave power and tidal power are low, the sediment of the river will build up long narrow banks on both sides of its course. Due to the gradient of the river flow, water levels at a fixed point along the river will gradually rise since this the distance of this point from the actual river mouth is increasing. At a certain moment, probably when the river discharge is high, the river starts overflowing the bank and it will erode a new shorter channel towards the sea. The same process is continuously repeated, which leads to an "elongate" or "birdfoot" delta. Strong waves with longshore currents tend to stretch the delta coast parallel to the general orientation of the shoreline, while strong tidal action usually creates patterns perpendicular to the shoreline. Outside the influence of the river, a strand plain develops when wave action is dominant and tidal flats develop when tidal action is the strongest.

In the transgressive case, an estuary is the equivalent of a delta in the prograding case, but now, the sediment supply is not enough to keep pace with the relative sea level rise. The sediment is no longer merely fluvial, but also has a marine source, since the flood tide or waves bring in sediment from the sea. A lagoon has a marine sediment source only, as no river is flowing into it.



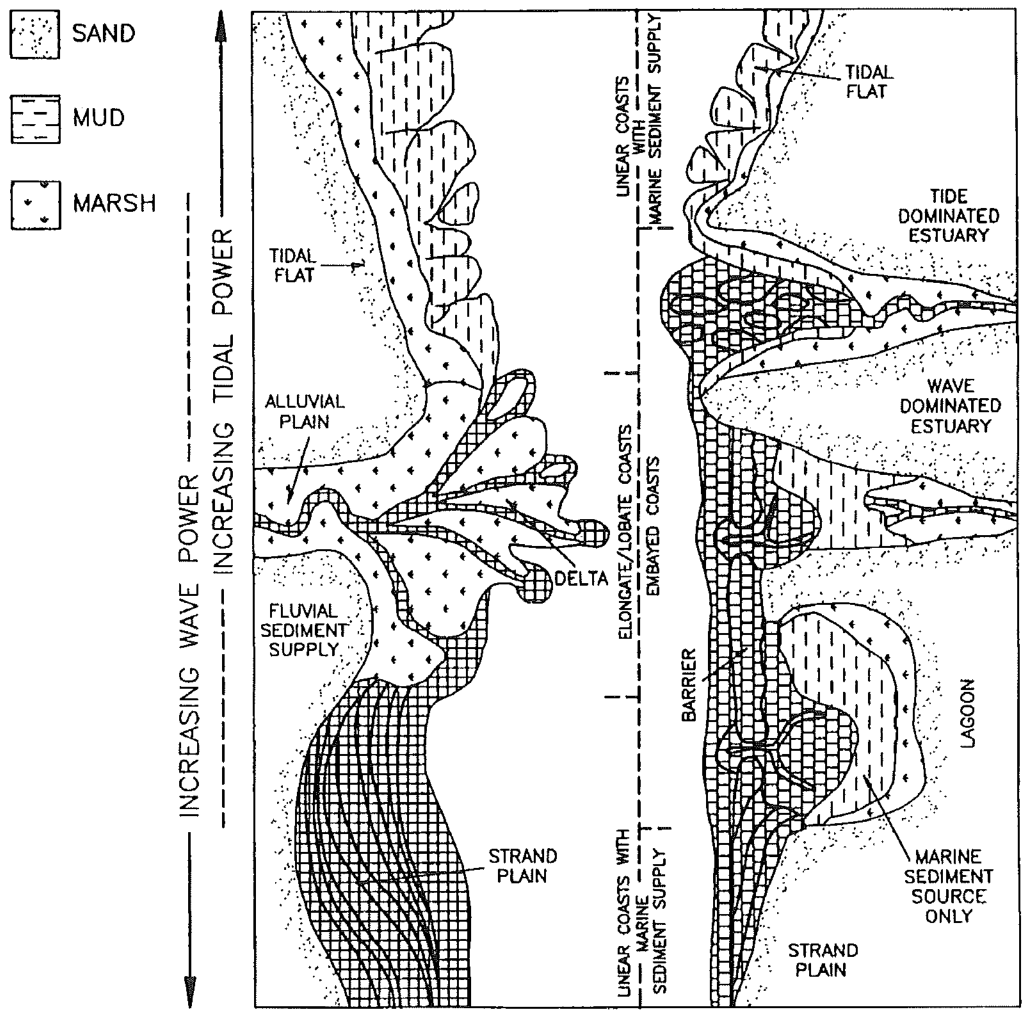


Figure 6-1 Coastal forms for prograding and transgressive coasts (Boyd et al, 1992)

Based on the various morphological processes, Figure 6-2 gives a classification for prograding and transgressive coasts. The ternary diagram presents the fluvial power on the vertical axis, and the coastal powers on the horizontal axis, wave power to the left and tidal power to the right. The top of the triangle represents deltas; the bottom strand plains and tidal flats; estuaries are situated in between. In this diagram lagoons form the end member of the estuary spectrum. The "depth" in the figure gives a possible idea of the evolution in time, relative to the change in sea level and sediment supply. With a rising sea level, all deltas change into estuaries and vice versa. Strand plains and tidal flats vanish and become shelf when the sea level rises.

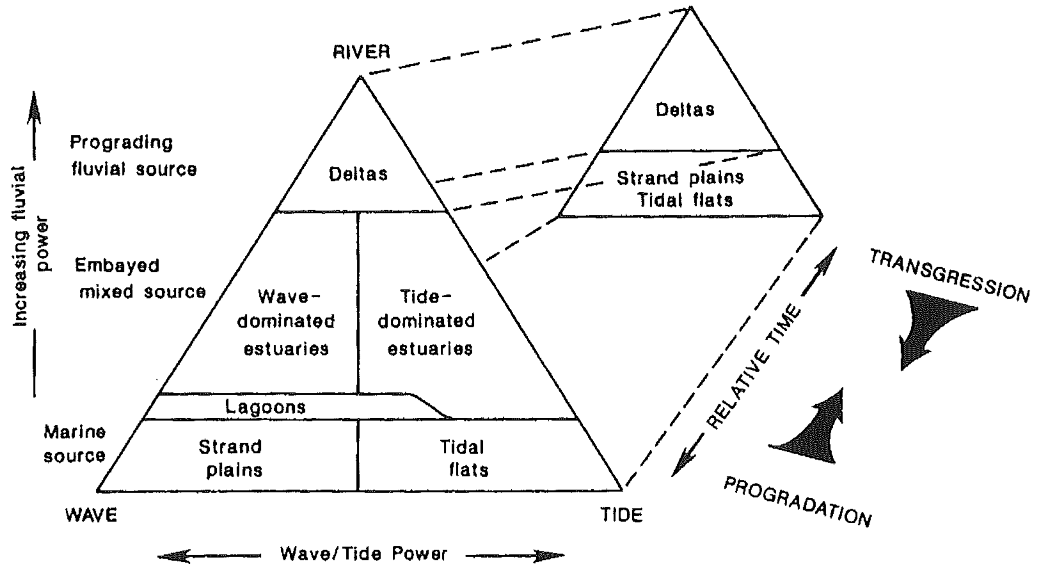


Figure 6-2 Ternary shoreline classification diagram (Boyd et al, 1992 and Dalrymple et al, 1992)

In the following section, different types of shoreline and shoreline elements in which sediment transport causes the typical shapes are discussed. These coasts can belong either to the transgressive type or to the prograding type. In some cases the distinction is not very clear without proper measurements.

Then, in Section 6.3, some types of coast in which biological influences play a dominant role in the development of characteristic shapes are considered. These biological influences can be related to flora or fauna.

Finally, in Section 6.4, typical features of rocky coasts will be treated.

6.2 Sediment dominated coastal features

6.2.1 Estuaries

An estuary is a tidal arm of the sea or part of a river that is affected by tides. It is the region in the vicinity of the mouth of a river where fresh and salt water mix. Estuaries form a dynamic environment, receiving fresh water from rivers, and salt water from the sea. Seen from the sea side, an estuary is an arm of the ocean that is thrust into the mouth and lower course of a river as far as the tide will take it. Every estuary has three main sections. The inland end, where the river enters, is called the head. The middle part is the fully estuarine area, where fresh water and salt water occur simultaneously. The seaward end is called the mouth.

Estuaries with wide mouths and narrow heads have a large tidal range. A tidal wave carries a given amount of water into an increasingly narrower part of the estuary. This geometry produces an increase in the tidal amplitude when the tidal wave enters the narrower upstream parts. An example of this effect can be seen in Canada's funnel-shaped St. Lawrence River. There, the tide increases in range from 0.2 m at the mouth of the river up to 5 m at Quebec City, a relatively remote part of the estuary located at the landward end. In the Netherlands also this can be observed in the Western Scheldt, although the difference between the tidal ranges in Flushing and Antwerp is not equally spectacular.

Seawater has a salinity of about 3.5 percent; fresh water has essentially zero salinity. This difference in salinity leads to different densities for the two types of water: 1000 kg/m^3 for fresh water and 1.025 kg/m^3 for seawater. More details about the salinity-density relationship are found in Section 4.2. This density difference plays a dominant role in the flow patterns in an estuary, and also in the behaviour of sediments. The principles of the density currents are explained in more detail in Chapter 9. At this point it is sufficient to state that the lighter river water tends to flow towards the sea near the surface, and that the heavier seawater is concentrated near the bottom when it enters the estuary during flood tide. Thus, with the tide, a salt wedge thus moves in and out the estuary. The angle of the interface between the fresh water and the salt water varies. If the angle of the interface is close to horizontal the estuary is termed stratified (layered) and if case the interface is close to being vertical it is called mixed. In Figure 6-3, stratification in an estuary is schematised.

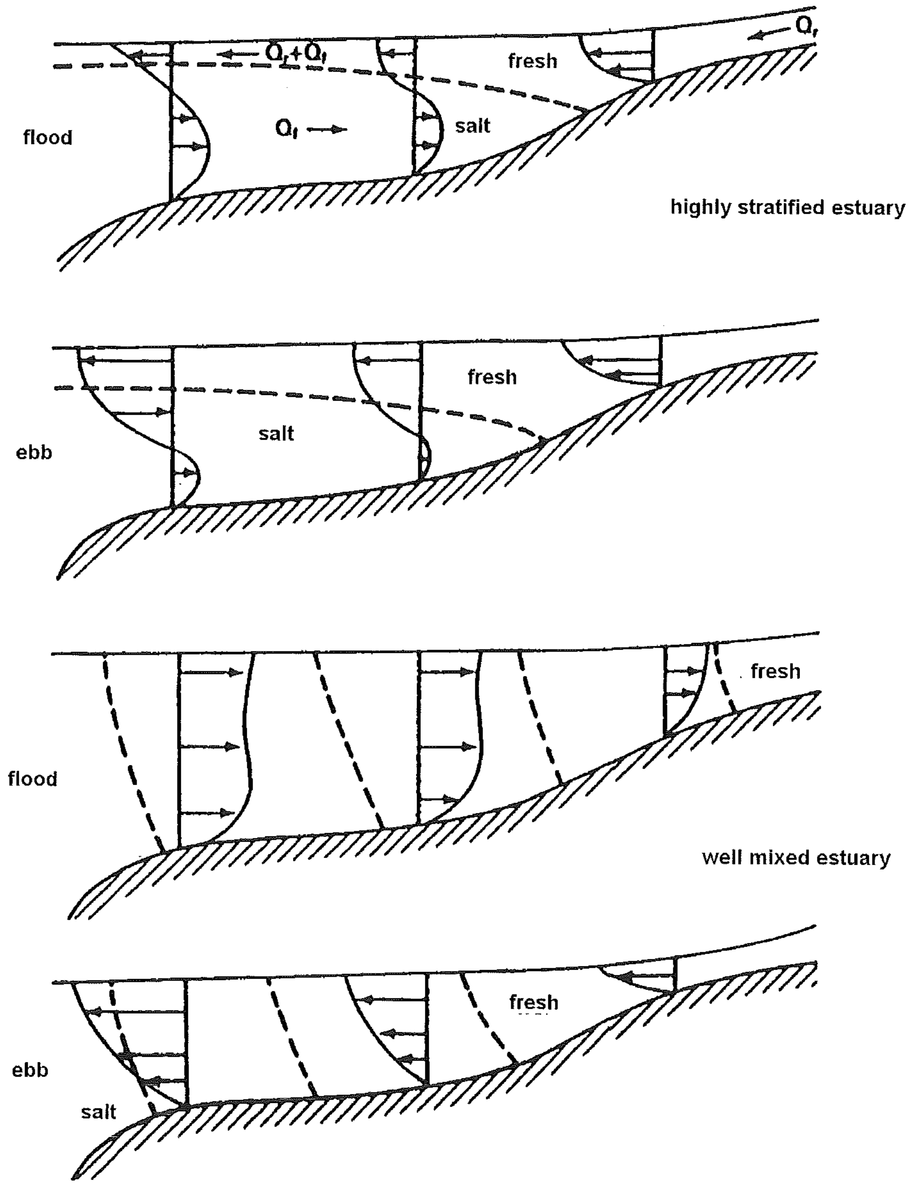


Figure 6-3 Stratification in an estuary: density variations and velocity profiles

The actual three-dimensional distribution of salinity in an estuary is very dynamic. It can vary in as short a time period as a single tidal cycle. It also varies with the seasons, and in relation to events like a storm surge at sea or a high river discharge.

The geologic lifetime of an estuary is short because of sediment deposition. Some estuaries fill so rapidly that they have a short life even in historical terms. When, in the late thirteenth century, Marco Polo visited the port of Hanghou (he called it Kinsai) on the Chien-tang estuary in northeastern China, it was a great commercial city with over a million inhabitants. Less than 200 years later, the bay had filled with sediment and trade had moved elsewhere.

River-carried sediment tends to be a mixture of sand, silt and clay, whereas sediments brought in from the sea generally consist of sand mixed with marine shells. Within the estuary, the finer fractions are typically transported as suspended load and sand is carried as bed load, rolling and bouncing along the bed of the estuary.

Within an estuary it is possible to distinguish three regions with a specific character on the basis of the driving force behind sediment distribution and deposition. We can, therefore, speak of river-dominated, tide-dominated, and wave-dominated zones in estuaries. The different processes in the estuaries are located differently, as can be seen in Figure 6-4. Figure 6-5 shows how the dominating processes characterise the estuary. In Figure 6-6, sediment transport processes averaged over time are situated in the estuary profile.

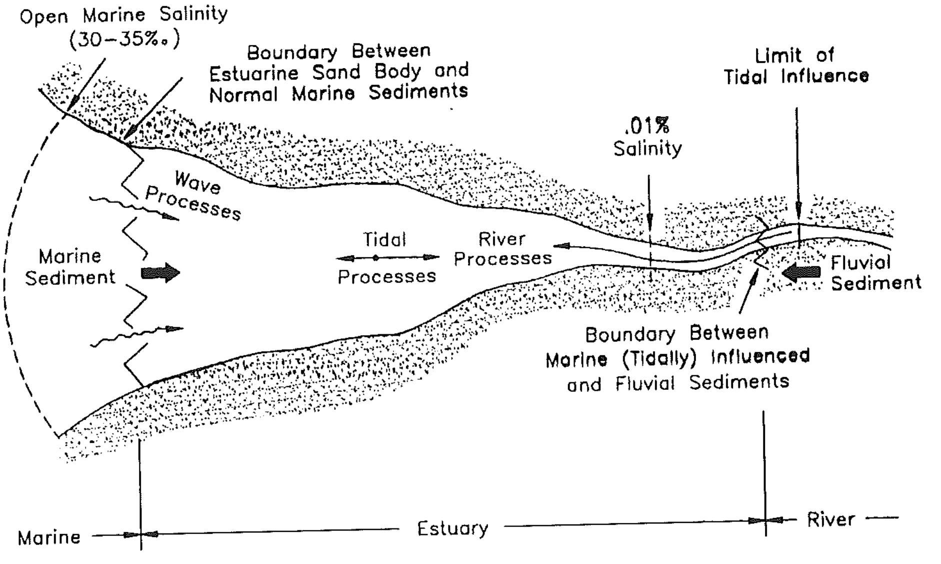


Figure 6-4 Plan view of distribution of energy and physical processes in estuaries

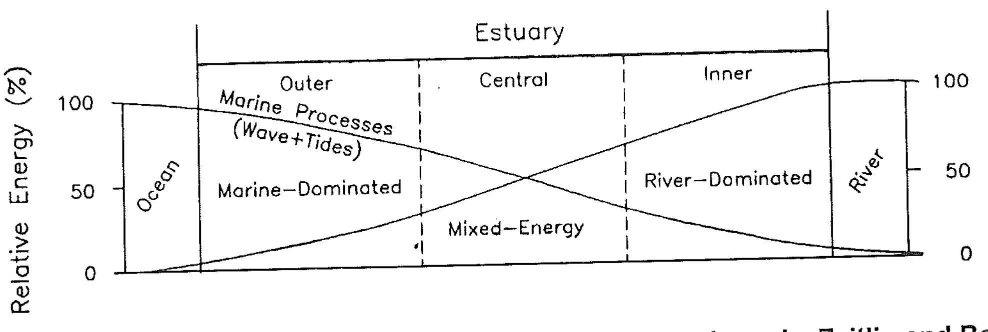


Figure 6-5 Schematic definition of estuary according to Dalrymple, Zaitlin and Boyd (1992)

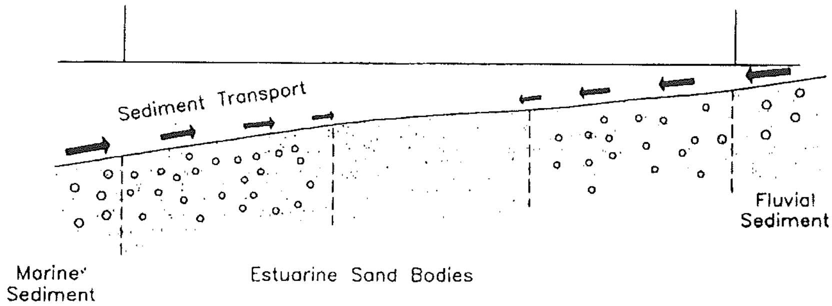


Figure 6-6 Time-averaged sediment transport paths

Coarse material like sand and gravel is typically transported as bed load. Finer particles, ranging from fine sand to silt and clay, are transported as suspended load. Because of the difference in fall velocity, sorting takes place in the estuary, with the coarser material settling first and the finer materials settling only in quiet areas. In the zone where fresh and salt water mix, the very small clay particles coagulate to form larger structures: *flocs*. Under the influence of the increased salinity, electrochemical forces bind the clay particles. Because of their size, the flocs have a higher fall velocity than the individual clay particles, which enhances sedimentation. Thus, large deposits of silt and clay are found particularly in the middle zone of estuaries, where fresh and salt water meet. Initially, the water content of the bottom layers is very high, and thus the sediment is very mobile and behaves as heavy water rather than as bottom material (sling mud).

Estuaries have another form of sediment in addition, which is contributed by the rivers and the sea, namely: the biogenic material produced in the estuary itself, by the estuarine population of plants and animals. Numerous organisms use the calm environment and the nutritious conditions to live permanently, to pass the winter or to spawn. Ostracods (tiny shrimp-like crustaceans in a bivalve shell), foraminifera, marine worms, and various snails, crustaceans, and bivalves are common estuarine animals. Shells and hard body parts from these organisms make an important contribution to the sediment of the estuary. Higher organisms like fish and birds live on the smaller animals and also leave much organic material in the estuaries. Therefore, estuaries form a very important element in the ecological structure of the coastal zone.

In industrialised and densely populated regions, rivers and estuaries have long been (and still are) used as sewers. The industrial waste usually contains considerable amounts of heavy metals and complex petro-chemical and other compounds that are bonded (attached by electrochemical forces) to the clay particles. Thus the deposits of mud in the estuaries are often heavily polluted and form a threat to the organisms living in the estuary. Because of the food chain within an estuary this forms a threat not only to the lower organisms, but even to human society.

Within an estuary, one can often distinguish a pattern of channels and gullies that separate the inter-tidal shoals. The deeper continuous channels are mainly responsible for the discharge of the river water and the ebb flow, shorter channels with dead ends are mainly flood channels.

6.2.2 Tidal flats

Large parts of most estuaries consist of tidal flats or wetlands. These are areas that are exposed at low tide and flooded at high tide. Their extent is determined by the shape of the estuary and by the tidal range. Obviously, a large tidal range will usually provide a broader inter-tidal environment under any given set of circumstances. Not so obvious is the influence of the slope of the shoreline along the estuary. Some slopes are very gentle and therefore provide wide tidal flats. However when slopes are steep for example, the sides of fjords or tectonically formed estuaries, the tidal flats, are narrow, even in a setting with a large tidal range. Much of the area of many estuaries all over the world is made up of tidal flats intersected by tidal channels.

The same currents that distribute sediments throughout the estuary and along the shoreline also deposit them onto the tidal flats. Local waves play a part, but most tidal flat systems are dominated by tidal currents. Tidal flat sediment is composed of mud and fine-grained sand and the shells of the small animals that have lived there; coarser grains settle out in the tidal channels. When exposed at low tide, the tidal flats have the appearance and texture of sandy mud or muddy sand.

As the tide ebbs and floods, the grains sort themselves according to size. The sediment on the tidal flat is deposited in thin, regular layers called tidal bedding. Each individual bed or stratum in this sequence can be from a few millimetres up to more than a centimetre thick. The tidal cycle leaves its own imprint on this bedding, producing alternating layers of sand and mud. Two sand layers represent the flood and the ebb portions of the cycle when currents are flowing rapidly. Thinner mud layers are deposited between the sand layers at, or near, slack tide, when fine sediment settles out of suspension. With the spring tides, neap tides, and storm tides also leaving their own specific record of sediment accumulation, it is sometimes possible to recognise hundreds of layers and reconstruct a coastal calendar of events. Geologists studying ancient stratigraphic records can recognise ancient tidal flats and tidal channels from their bedding characteristics and can even reconstruct the behaviour of tides.

6.2.3 Deltas

Classification of deltas

Deltas are typically connected to a prograding coast, while the opposite is true of estuaries. However, it is possible to find combinations of the two, when over the geological history progradation and transgression have alternated. The south west part of the Netherlands prior to the execution of the Delta project was an example of such combination, within the delta of the River Rhine, estuaries like the Haringvliet and the western Scheldt could be distinguished.

In Section 6.1 it has been shown that deltas are formed where a river carries large quantities of sediment and deposits them in the sea. Thus, deltas are transitional coastal environments that are neither fully terrestrial nor fully marine. They have no easily recognisable landward or seaward boundaries, but change by almost imperceptible stages from open sea to solid ground. A delta begins at the point where a large, sediment-laden river leaves its upland drainage basin and flows onto a region adjacent to the ocean. Built primarily from river-borne sediment, deltas form when the amount of sediment delivered at the mouth of a river exceeds the amount removed by waves and tidal currents.

Like estuaries, although their "morphological opposites", deltas are strongly influenced by rivers, waves, and tides. Their influence determines the shape and the character of each estuary to a large extent. William Galloway's triangular diagram classifies deltas according to the relative influence of these three major factors affecting their development. It is given in Figure 6-7.

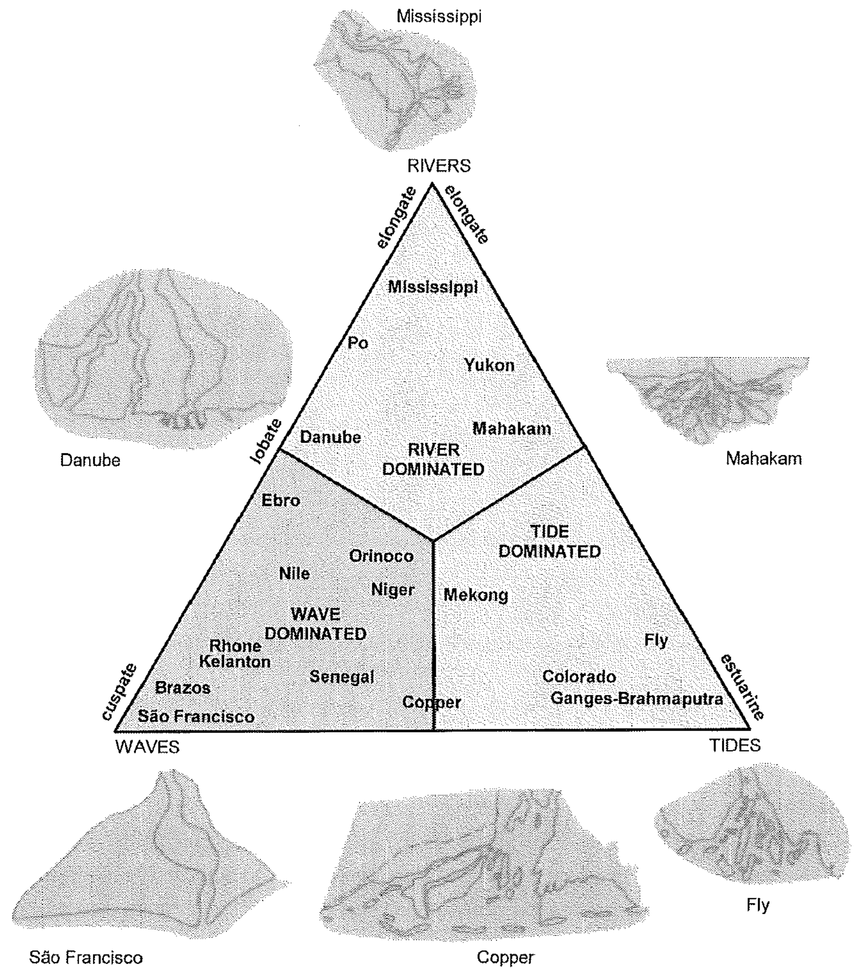


Figure 6-7 William Galloway's triangular delta classification diagram

Shapes of deltas

The formation of a delta depends on the interaction between the flow and distribution of the river sediment, the waves and tidal currents. As the water flows from the river mouth, its velocity decreases and it loses its capacity to carry sediment. Consequently, sediments accumulate in the river mouth area. As the velocity of the out flowing river water decreases, the coarse material settles first, followed by the finer sediments. At the seaward edge of the delta front, the suspended sediment in the river water finally settles out into the deeper coastal water. This mud accumulation is generally very thick and extends across part of the continental shelf.

The amount and configuration of sand accumulating in the delta-front depends on the relative roles of the interacting river, wave, and tidal currents. A common type of sand accumulation is a sandbar that forms just seaward of the channel mouth and typically causes the channel to bifurcate. Another is the formation of banks along the sides of the channel. As the river deposits sand in the mouth, the situation can be reached in which the water level is affected by the sand deposits. The river can then overflow its banks and then divide into *distributary* channels. Each distributary channel then continues to transfer massive amounts of fine-grained sediment to the coastal area. When this new-born delta is situated in an environment with little tide and wave action, it is categorised as being <u>river dominated</u>. It can grow out into a <u>birdfoot</u> type of delta. Examples of this type of estuary are shown in Figure 6-8 and Figure 6-9 (Mississippi River and Danube Delta respectively).

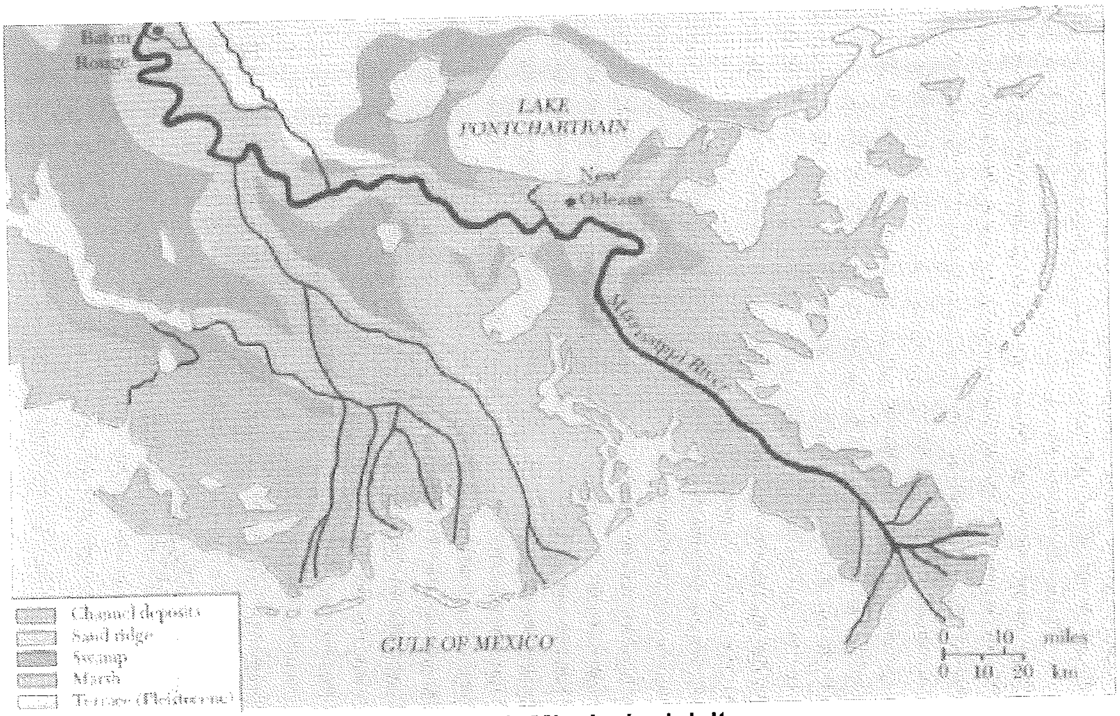


Figure 6-8 Mississippi delta

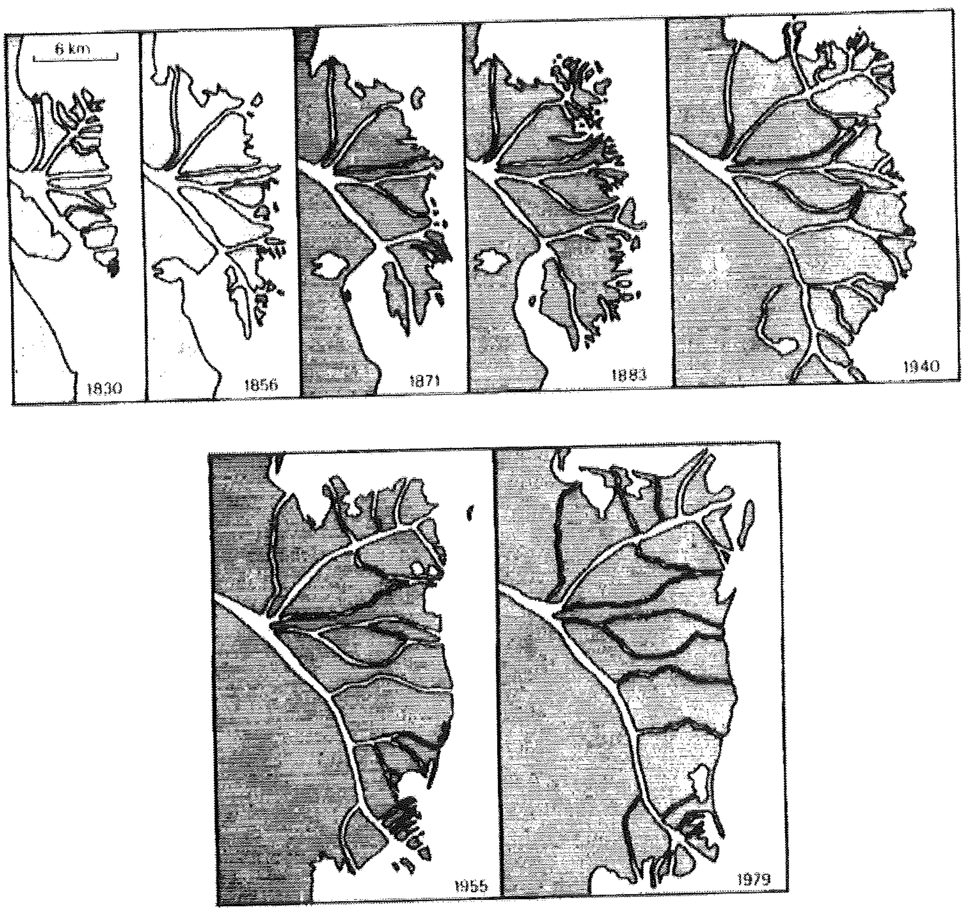


Figure 6-9 Historical stages in the growth of the Kilia Lobe of the Danube delta, Romania (Bird, 1984)

Tide-dominated deltas develop where a large range between the high and low tides leads to strong tidal currents. On these coasts, the wave height is moderate to low, and longshore currents are weak. These deltas resemble estuaries because of their embayed setting of salt marshes, swamps, and tidal flats. An example of such tide-dominated delta is the delta of the river Fly on the South coast of Papua New Guinea (Figure 6-10).

Figure 6-10 Delta of the river Fly, Papua New Guinea

If the wave climate is more severe, the bars at the river mouth are affected by the waves. As a result of the wave action, sand is re-positioned by longshore and cross-shore effects. Depending on the wave direction, this can lead to a delta that is symmetrical or asymmetrical in shape. The symmetrical case (with waves perpendicular to the shore) is indicated schematically in Figure 6-11.

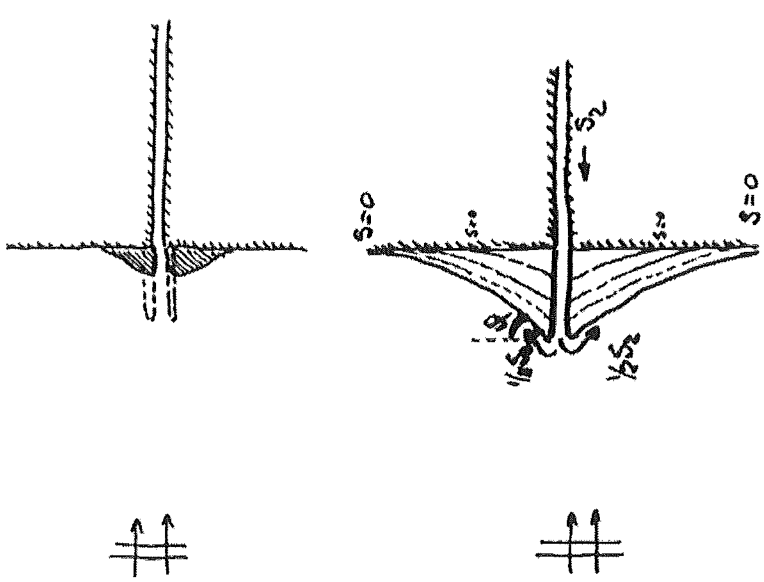


Figure 6-11 Formation of a wave-dominated delta

Wave-dominated deltas typically have a rather smooth shoreline with well-developed beaches and dunes. The delta plain tends to have few distributaries; some deltas of this type have only a single channel. A wave-dominated delta is generally smaller than other types, because the distributing power of the waves striking the delta front is stronger than the carrying power of the river. When the wave climate is strong enough to carry all the river sediment away, the delta will shrink and eventually disappear. Two different shapes characterise these deltas. The general shape is symmetrically cuspate. One of the best examples is the delta of the Sao Francisco in Brazil (Figure 6-12). The other shape is characterised by a strong longshore current. A sand spit develops and protects the extensive wetlands that cover the delta plain. An example of this is the Senegal River Delta, as can be seen in Figure 6-13, or the Ebro delta (Figure 6-14).

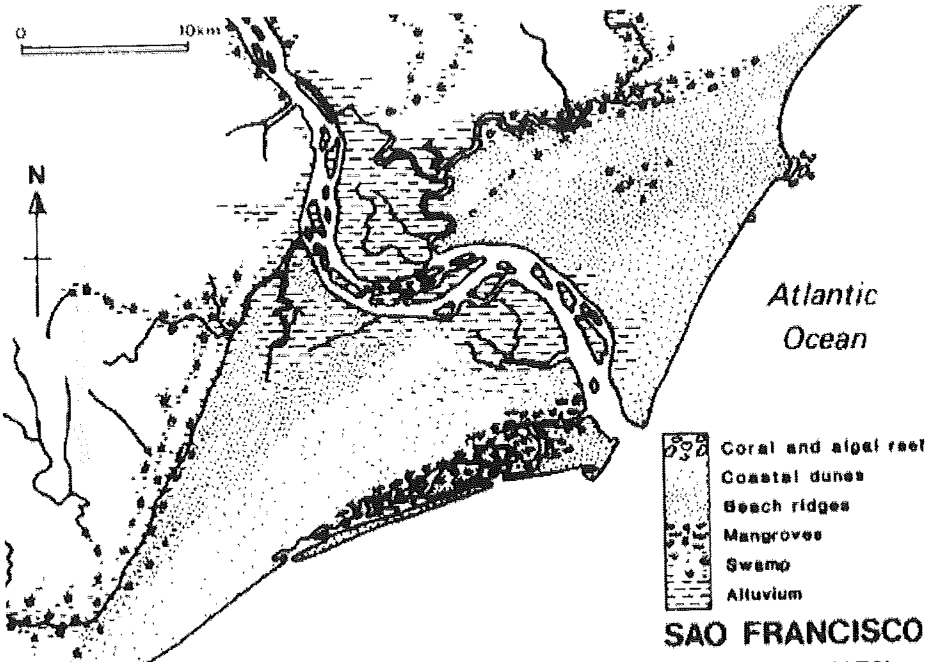


Figure 6-12 Sao Francisco delta, Brazil (Wright and Coleman, 1972)

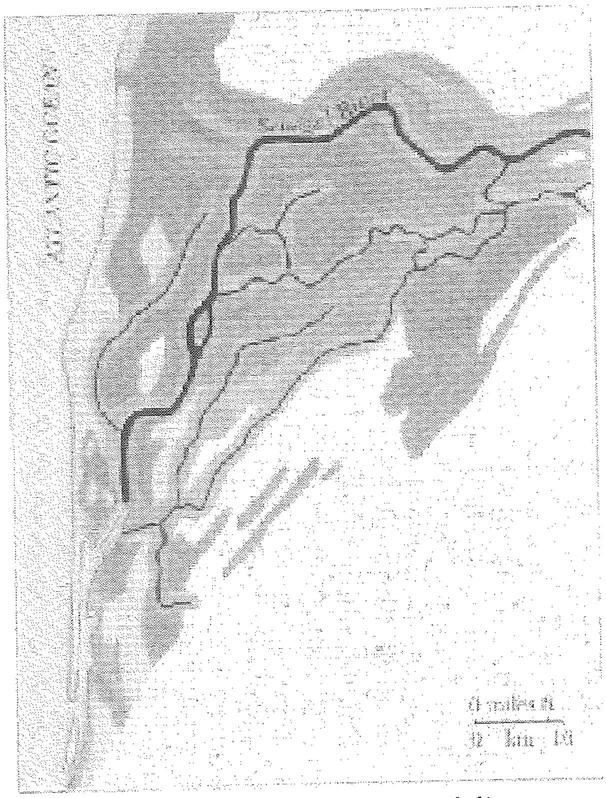


Figure 6-13 Senegal river delta

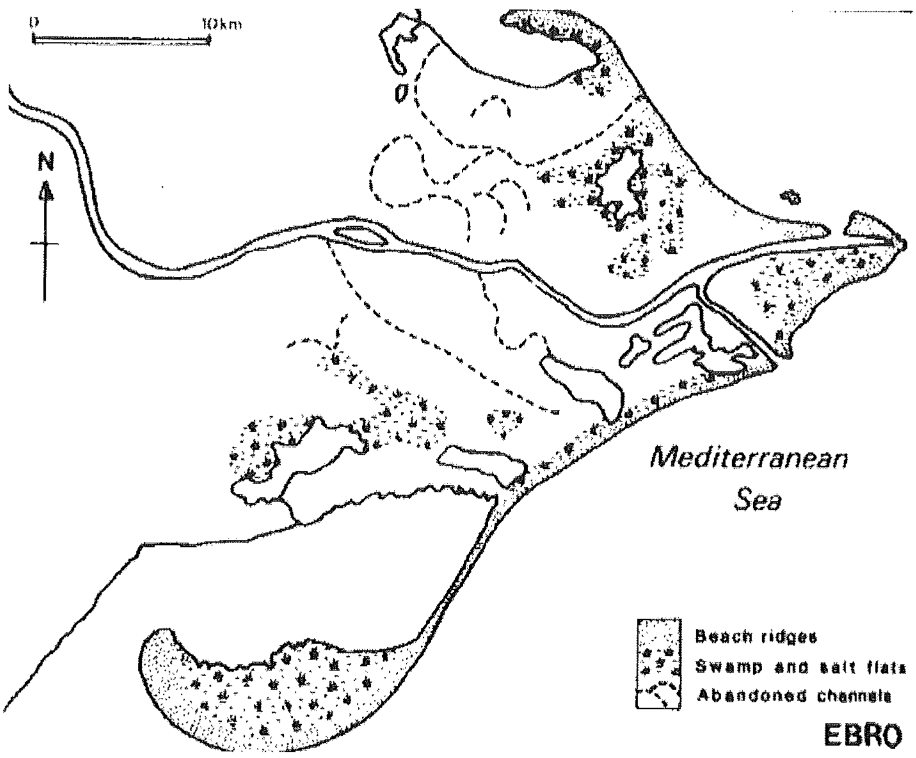


Figure 6-14 Ebro delta, Spain (Wright and Coleman, 1972)

Resulting landscape

The landward and very flat part of a delta is the delta plain (Figure 6-15). The upper delta plain is merely an extension of the upland meandering river system, except that the river here consists of one or more distributary channels. Each time a distributary channel overflows its banks, the coarser sandy sediment particles are dumped first, producing a low ridge of accumulated sediment along the bank margin. This ridge is the natural levee. It may build up to an elevation of a meter or two above the surrounding delta plain. During subsequent flooding, the natural levee may be breached either through a naturally low section or through cuts made for human passage. When the sediment-laden floodwaters pass through the breach, generally called a crevasse, there is an immediate reduction in carrying capacity as their velocity decreases abruptly. A thin, fan-shaped sediment accumulation forms beyond the breach. This formation, called a crevasse splay, can extend several kilometres across the upper delta plain.

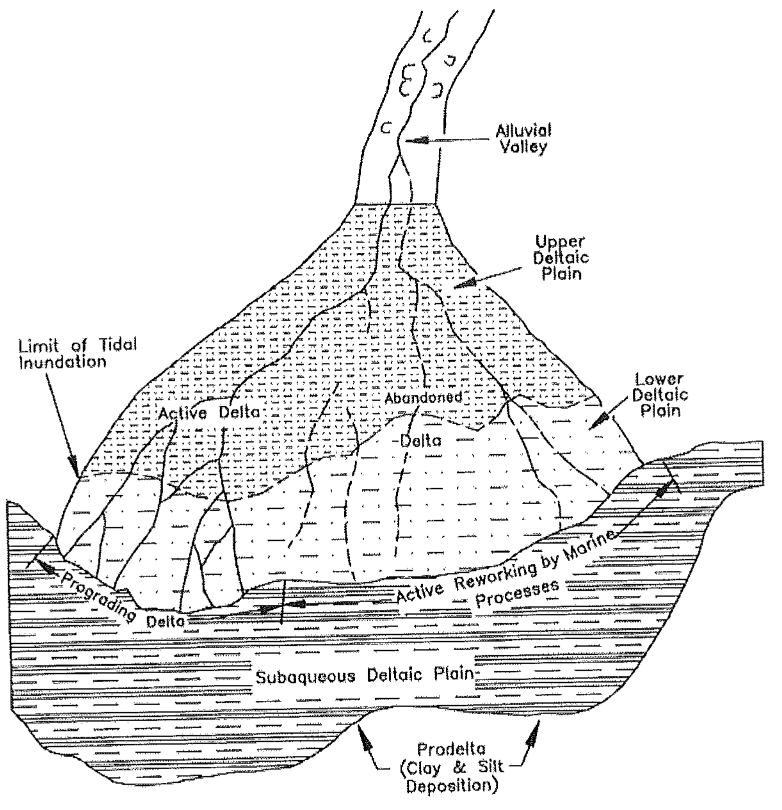


Figure 6-15 Basic environments of a delta

The major landforms of the delta plain - natural levee, crevasse splay, inter-distributary bay and marsh - are distinguished from one another on the basis of elevation, sediment character, and vegetation. As time passes, continued flooding and sediment unloading enlarge the delta and bring more and more of its features above water level. Much of the mature delta plain between the distributary channels eventually turns into fertile farmland, interspersed with small lakes and fresh water marshes and swamps. All these are periodically replenished by flood waters. The inside edges of the bends in the distributary channels on a delta plain fill with thick accumulations of sand and gravel. These deposits are called point bars. As the channels migrate across the delta plain, they leave subtle but recognisable scars marking their former locations.

It is this mixture of fertile land, fresh water and seawater with all its gradients that creates an almost ideal habitat for any form of life. The abundance of natural resources and the tremendous bio-diversity have turned deltas all over the world into the most densely populated zones. At the same time, this dense population forms the major threat to their sustainability.

Relation with geology

Deltas occur on every continent and in a wide range of climatic settings, but the geological settings are generally similar. A tectonically stable trailing edge coast provides the right conditions for delta formation. It has low to moderate relief terrains, such as coastal plains or geologically old mountain areas. Rivers bring an abundant sediment supply across wide, gently sloping land, where the river channels meander back and forth on their way to the coast. On the seaward side of this tectonic setting, the broad continental shelf provides a platform suitable for sediment accumulation; it also reduces the size and energy of the incoming waves.

The Sao Francisco Delta in South America and the Senegal Delta in Africa have developed on trailing edge coasts. Marginal seas with trailing-edge characteristics provide shelter from large waves and tides, and very large deltas have developed in tectonic settings of this type. Excellent examples are: Mississippi River Delta in the Gulf of Mexico; the Rhone, Nile, Po, and Ebro Deltas in the Mediterranean Sea; and the huge deltas of China that empty into the South China Sea.

Most of the present active deltas are geologically very young features; some are only a few hundred years old. Because a delta develops at the coast, its existence is, in part, controlled by the sea level. It therefore was and still is, vulnerable to (global, eustatic) sea-level rise, too. During the periods of extensive glaciation, sea levels were much lower and rivers traversed the present continental shelves, dumping their sediment loads at or near the outer shelf edges. This suspended sediment cascaded down the continental slopes in turbulent, high-density flows called turbidity currents. New deltas did not form during this period, and deltas that had previously existed near the positions of present-day coasts were abandoned and entrenched by rivers as they flowed across the continental shelves.

Melting glaciers brought a rapid rise of sea level, and river mouths retreated so rapidly that deltas could not develop. Finally, about 7000 years ago, the Holocene sea level rise slowed, and in some parts of the world it stabilised at approximately its present position. Where conditions were appropriate, deltas began to develop as large quantities of river sediment accumulated.

Not all present-day deltas are only up to a few thousand years old. Many of them have formed on ancestral deltas built up during previous interglacial periods. A few, such as the Mississippi (Figure 6-8) and Niger Deltas, are underlain by ancestral deltas that formed tens of millions of years ago. The upper regions of these mature deltas are also ancient, but their active delta lobes are only between 3000 and 6000 years old. The lower Mississippi Delta includes 16 detectable lobes. A new lobe forms whenever the location of the river mouth changes. The channels of abandoned lobes fill up with sediment, contributed both by the river, by the waves and by the tides of the coast. The present delta lobe of the Mississippi dates back only 600 years; its most active portion has developed since New Orleans was founded in 1717.

6.2.4 Beaches

Nearly any type of non-cohesive granular material that can be transported by waves can form a beach. A beach extends from the low tide line landward across the un-vegetated sediment to the beginning of permanent vegetation, or to the next geo-morphologic feature in the landward direction - a naturally-occurring dune, a rocky cliff, or a constructed seawall. The overall profile of a beach and the adjacent near-shore depends on sediment supply, wave climate, overall slope of the inner continental shelf, tidal range, and a variety of local conditions. Sandy beaches include a foreshore and a backshore (Figure 6-16). In many places, a pair of persistent sandbars, over which waves break during storms, parallels the beach.

The backshore, or back-beach, extends from the berm at the landward end of the foreshore across the remainder of the beach. Gravel beaches of shell and rock fragments commonly include a storm ridge that is just landward of the foreshore. Sometimes this storm ridge may grow until it rises several meters above high tide and entirely replaces the back-beach. Its composition depends on the nature of the gravel material in the immediate area; its size is proportional to the rigor of the storms that produce it.

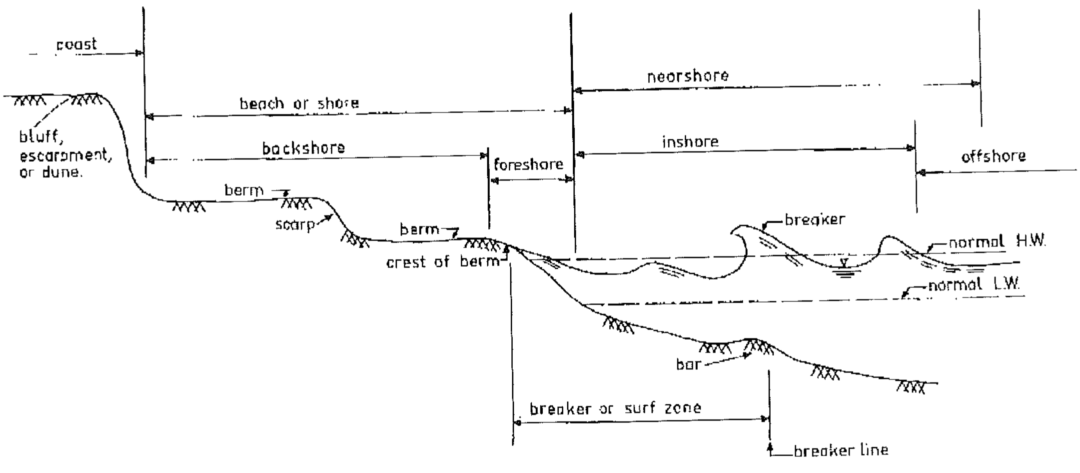


Figure 6-16 Sandy beach profile nomenclature (distorted scales)

In Chapter 5, morphological processes causing beach change were mentioned. Many of these are cyclic processes. Their scale of time and space can differ very much from one process to the other. Cycles can be long-term or short-term. For example, the season causes beach variation. Depending on the climate, on one hand there are the fair-weather, low-energy, accretional beaches and on the other hand foul-weather, high-energy, erosive beaches. The correlated beach profiles are called summer and winter profiles.

Special beach forms are the tombolo and the spit. They are formed due to the longshore transport of sand along the beach. In case of a tombolo, the driving force behind the longshore current, (i.e. the waves), is interrupted by an offshore island. Due to the reduced transport capacity sand settles in the lee of the island and forms a typical outcrop on the beach, which may eventually even connect with the island (Figure 6-17).

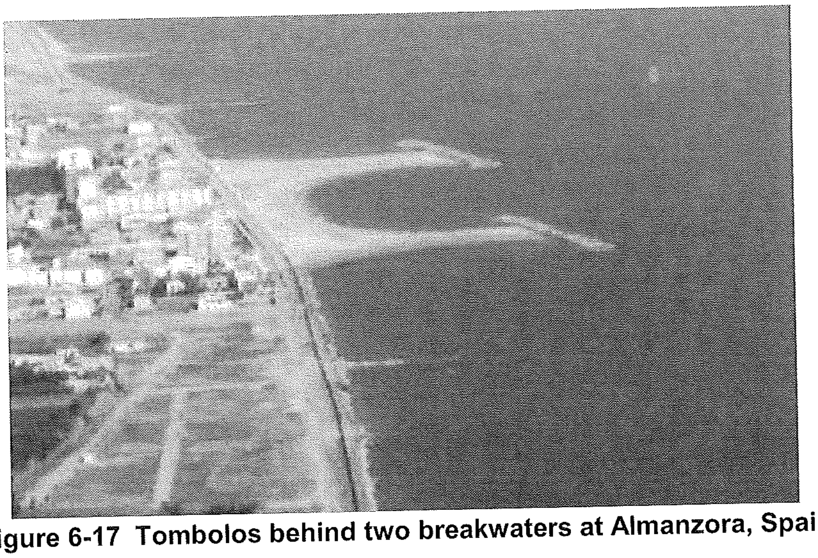


Figure 6-17 Tombolos behind two breakwaters at Almanzora, Spain

A spit forms at the end of a beach, where the longshore current looses its transport capacity. The sand carried to the end settles in deeper water and gradually it forms a ridge, that is more or less as an extension of the beach (Figure 6-18).

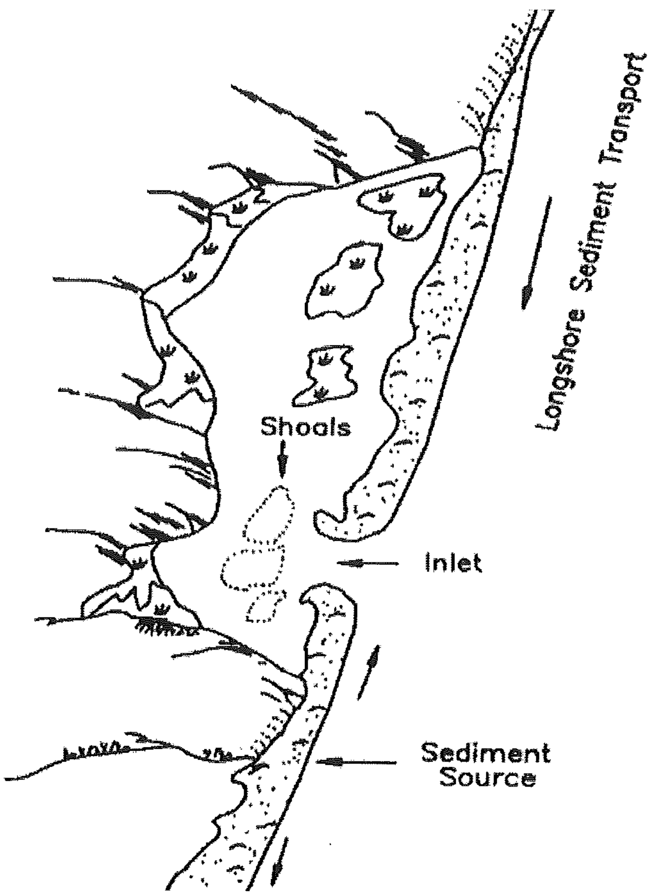


Figure 6-18 Spit

Beaches occur, in both conditions of transgression and of progradation.

6.2.5 Dunes

Dunes can either be bed forms (typical shapes in the seabed) larger than ripples and smaller than bars or ridges or, above water, mounds of loose wind-blown material such as sand. Both play a role in coastal engineering. In the present context we confine ourselves to the wind-blown mounds that are quite familiar as the landward boundary of sandy beaches. The dunes often provide the stockpile of material that allows the beach to adapt to changing seasonal or incidental conditions, specifically when these conditions create a more gentle foreshore profile or a higher base level of the foreshore due to sea level rise or storm surge.

In developing dunes, the prevailing winds or diurnal sea breezes provide the transport mechanism. Vegetation is one of the best and most widespread facilitators of dune development. Different dune types are shown in Figure 6-19. Dunes can be two or three dimensional (Figure 6-20).

A dune ridge - a linear arrangement of dunes, one dune wide - is the typical configuration of dunes just landward of the beach. It is called the foredune ridge because of its location seaward or in front of the barrier or mainland.

As mentioned earlier, dunes often serve as a sand reservoir. Although dunes are beyond the regular influence of waves, they are vulnerable even to modest storm surges. The other major disturbing factor is the wind, which can cause migration of all or part of the dune. Blowover is the most common wind-driven process responsible for dune migration. It is effectively stopped by vegetation that can survive the harsh growing conditions. Protection of such vegetation (or even artificial cultivation) is one of the technical measures used to stop sand migration.

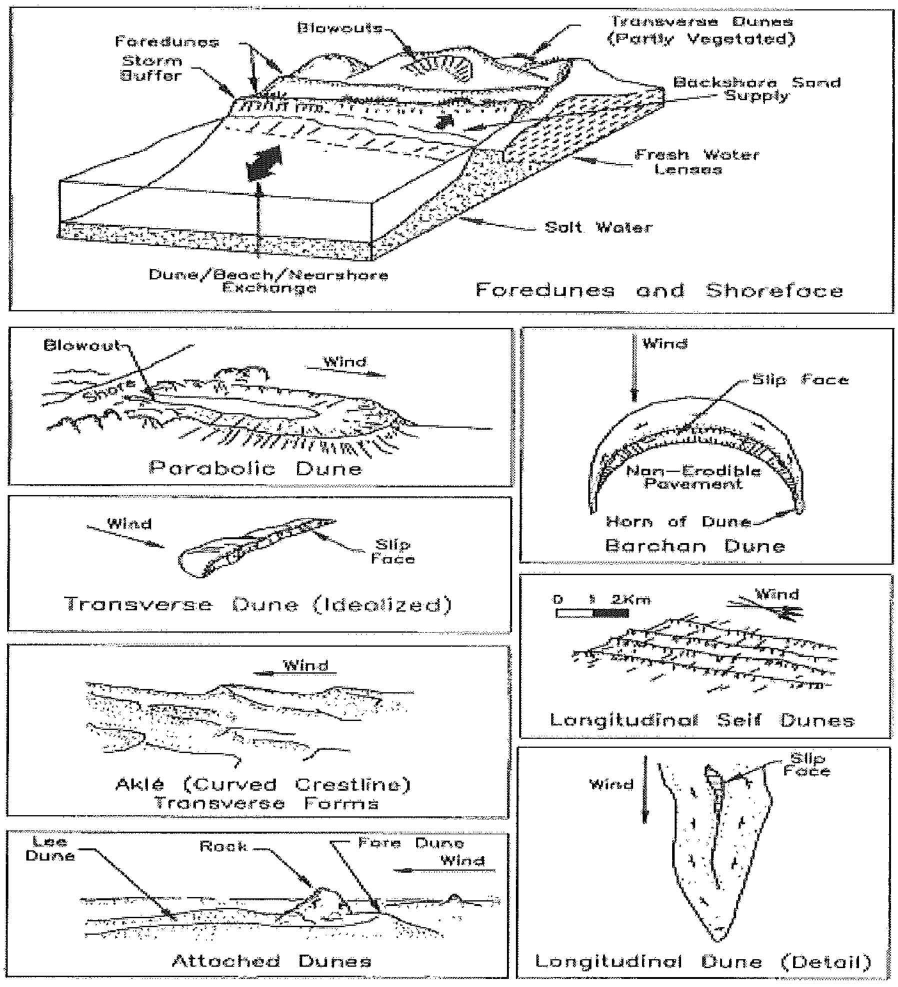


Figure 6-19 Variety of Dune Types (Carter, 1988, Reading, 1986 and Flint, 1971)

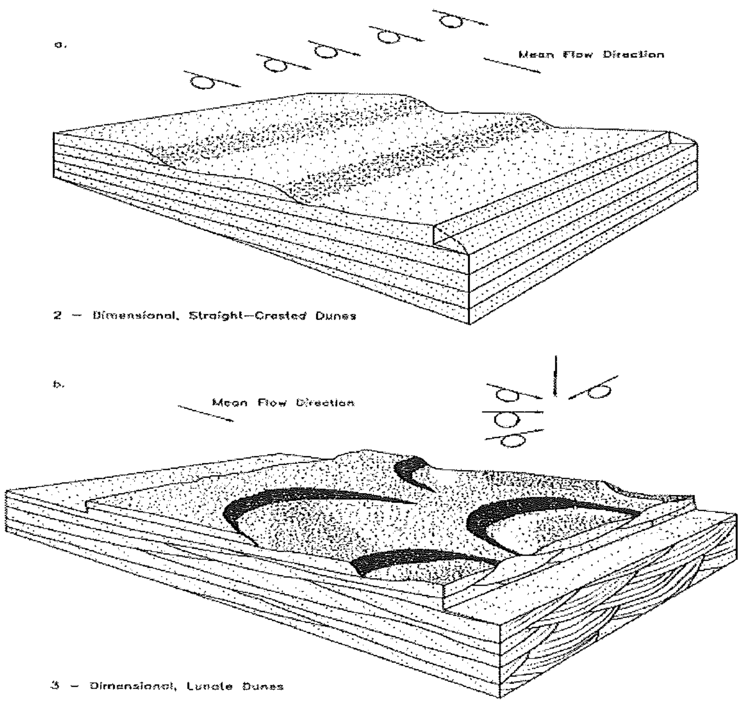


Figure 6-20 Two dimesional and three dimensional dunes (adapted from Reineck and Singh)

An important aspect of the presence of dunes in the marine environment is the capability to accumulate and store fresh water. Since the dunes have a certain elevation, they will catch the first rains from the ascending sea winds when they pass over the elevated surface. Due to this rainfall a local groundwater level will be established in the dune that is *ΔH* above Mean Sea Level. The pressure at a depth H below Mean Sea Level is than: $\rho g(H+\Delta H)$, with ρ being the density of fresh water, i.e. 1000 kg/m^3. The pressure in the underlying salt water at the same level must be equal. So:

$$\rho_{fresh\ water}\, g(H+\Delta H) = \rho_{seawater}\, g H \tag{6.1}$$

After a bit of mathematical calculation this becomes:

$$H = \frac{\rho_{fresh}}{\rho_{sea}-\rho_{fresh}}\Delta H = 40\Delta H \tag{6.2}$$

In words this means that a slight elevation of the water table of the fresh ground water within the dune causes the development of a huge fresh water lens under the dune, provided of course that sufficient fresh water is available to fill this lens. In this way, a valuable fresh water source is available close to the marine environment. In many places, this dune water has been used as source of good quality drinking water, merely by pumping it up. A consequence of this water withdrawal is that the watertable in the dune will become lower, and that the interface with the saline deep ground water is rises. This has two effects: the source will gradually turn saline, and the lowering of the watertable will endanger the vegetation. Disappearance of the vegetation will cause erosion of the dunes by wind. (See also Section 6.3).

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6.2.6 Lagoons

Lagoons (Figure 6-21) are bays that are closed off from the sea in the sense that there is no important tidal influence inside them. Lagoons can be seen as a specific type of estuary.

Generally, they are protected from the open sea by a barrier island, a reef, or an obstruction that prevents wave attack and inhibits tidal circulation. Stages in the evolution of a barrier to enclose a lagoon are shown in Figure 6-22. The prolongation of the spit, as is shown in the upper part of the figure, is caused by the longshore transport. Shoreward migration of a barrier that originated offshore is caused by the cross-shore transport. It is shown in the lower part of the same Figure.

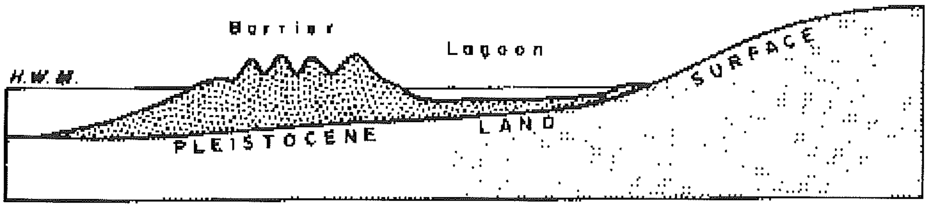


Figure 6-21 Section through a Barrier closing a Lagoon (Bird, 1984)

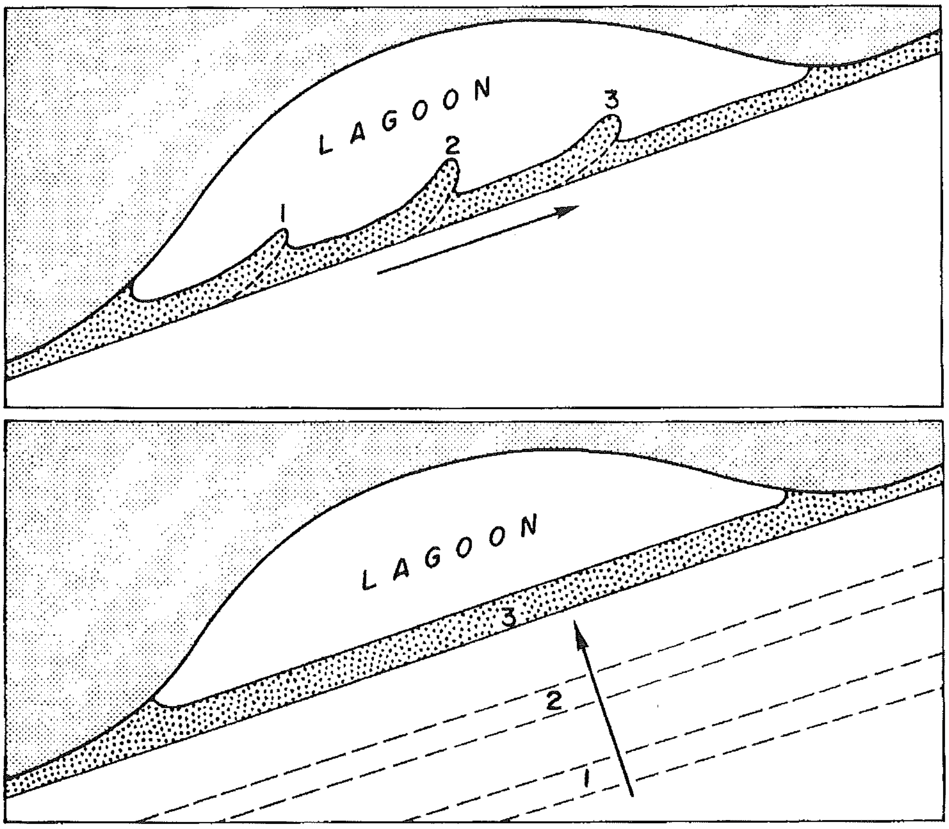


Figure 6-22 Stages in the evolution of a barrier to enclose a lagoon (Bird, 1984)

Usually, there is no large freshwater influx in a lagoon. Sometimes, a small river discharges into the lagoon, so the water becomes brackish or fresh. This cannot be a large discharge however, otherwise the connection between the lagoon and the sea would change into a tidal inlet including all correlated dynamic processes. The typical lagoon configuration would then vanish unless the lagoon is situated along a coast with very small tidal amplitude (Baltic Sea or Mediterranean).

In a lagoon, specific morphological processes are at work. They are related to the protected nature of the area that forms an ideal boundary condition for and include the formation of peat and settlement of very fine sediment.

6.2.7 Barrier coasts

A barrier can be defined as an elongate, shore-parallel sand body, which may consist of a number of sandy units including beach, dunes, tidal deltas, wash-overs, and spits. Barriers separate lagoon and estuary embayments from the marine environment and are best classified as components of estuary and lagoon systems. General barrier types are given in Figure 6-23. Barriers rise above sea level, naturally protecting the landward part of the coast against wave attack.

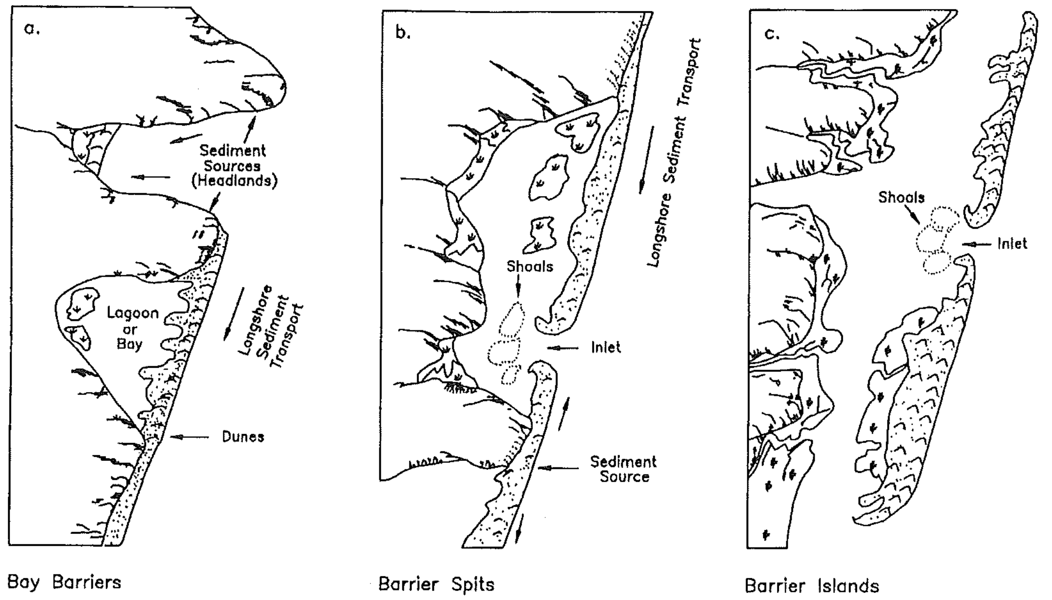


Figure 6-23 General barrier types: bay, spit, island

Barrier islands develop in any geological and tectonic setting that has a plentiful supply of sediment, agents to transport it, and a site where it can accumulate. Therefore, barrier islands and other barriers are often found along trailing edge margins, as well as on scattered parts of leading edge margins. They form as sand accumulates through the combined action of waves and wave-generated long-shore currents.

Barrier islands can range from a few hundred meters to more than 100 km in length. They can be defined as wave-dominated and mixed-energy deposition systems. They are found worldwide, from Alaska to Australia; they constitute approximately 12 to 15 percent of the earth's outer coastline. Some are barely above high tide; others have dunes that rise 30 m above the sea.

Whether a barrier island develops at all depends on the predominance of wave over tides. Regardless of its specific origin, waves and wave-generated currents must be present to produce the linear accumulation of sediments. When tides become dominant over waves, the barrier island gives way to a tidal flat and marsh.

Barrier islands tend to "walk". If deposition of storm-transported sediment continues over an extended period of time at the back of a barrier island, a landward displacement of the barrier occurs. As the barrier transgresses, it loses some of the sand-sized sediment. Unless more sand becomes available, largely through longshore transport, the barrier is destroyed before it reaches the mainland. Sea-level rise makes barriers more vulnerable to this type of movement.

Addition of sediment can also cause barrier islands to progress, that is, to grow in a seaward direction. This condition is different from transgression in the sense hat the barrier island as a whole does not move. Instead, the addition of sediment causes the development of multiple beach-dune systems, and the open water shoreline actually moves seaward, while the landward backbarrier shoreline remains in place. An individual barrier island can experience transgression and progradation at the same time.

Under the influence of longshore transport, a barrier island can also move parallel to the coast. Material eroding from one tip will eventually settle at the tip of the adjacent island.

6.2.8 Tidal inlets

Barriers generally are breached at various points by tidal inlets (Figure 6-24), which link the open marine environment and he coastal environments landward of the barrier islands. Like beaches, tidal inlets are dynamic parts of the barrier system and range widely in size, stability, and water flux. They owe their origin to a variety of circumstances, although storms and human activities are the most important factors. Flood tidal deltas and ebb tidal deltas either can be tide-dominated or wave-dominated. Another important factor is the bathymetry of the back-barrier bay. Along many barrier coasts, for example the Dutch Wadden Coast, the longshore transport causes a structural asymmetry in the ebb-tidal deltas.

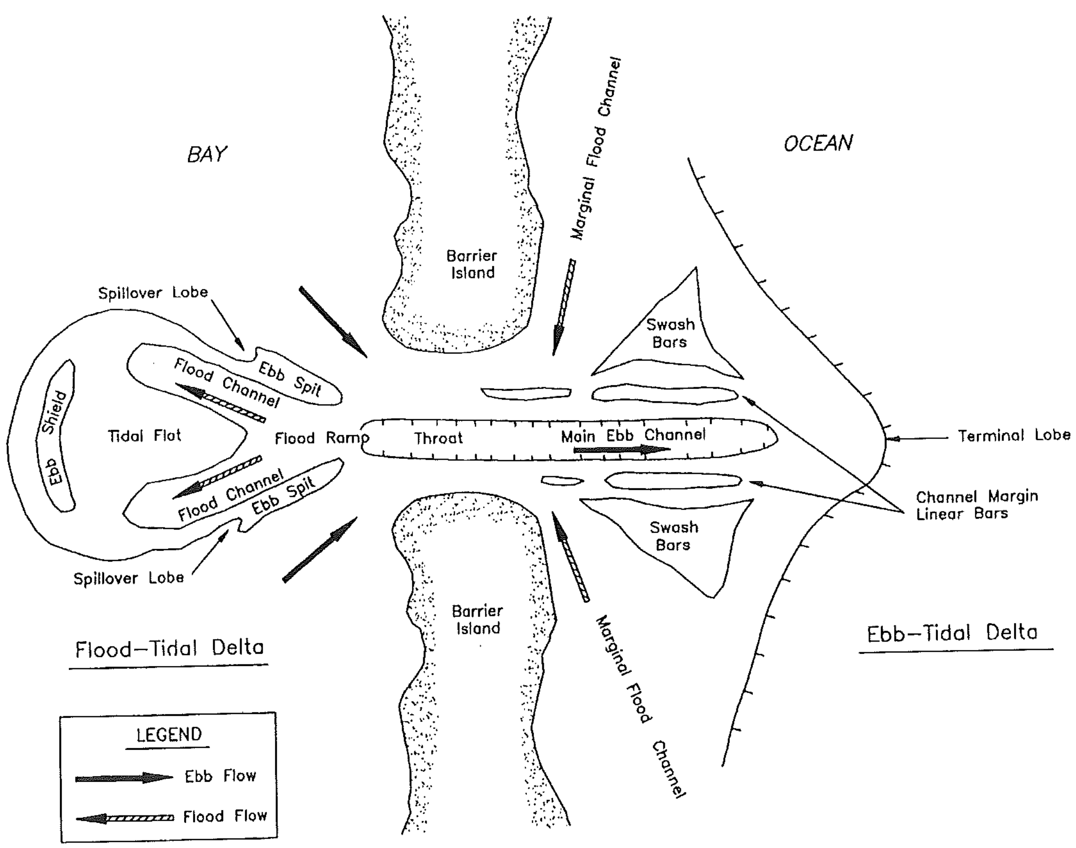


Figure 6-24 Geological model of a tidal inlet (Boothroyd et al, 1985)

6.3 Biology dominated coastlines

6.3.1 Salt marshes

On the fringes of estuaries, lagoons, and other bays, in places where sediment deposits are sheltered from wave action and high enough above sea level, salt resistant vegetation gets a chance to establish a foothold. When conditions remain favourable, the number of species increases and the plant cover gets denser. The roots stabilise the bed material, and the stems and leaves reduce the current velocity near the bottom. In this way, conditions for further sedimentation are enhanced. By the rising level of the flat, the conditions become more favourable for other species. As the bottom level rises, the salinity decreases because of the better drainage of the salt ground water and the growing influence of precipitation.

The type of vegetation depends strongly on climatic conditions, the composition of the sediment and the salt content of the water. Specifically the silt content is an important factor. In the moderate climate of the southern North Sea, most vegetation occurs in silt rich areas. Pioneer species in the salt environment are cordgrass or "Engels slijkgras" (Spartina Anglica) (see Figure 6-25), and in more brackish conditions sea club rush or "Zeebies" (Scirpus maritimus), various sea-grasses (Zostera marina and Zostera noltii). Later on, "Nopjeswier" (Vaucheria) and glasswort or "Zeekraal" (Salicornia stricta) become established, immediately followed by a greater variety such as herbaceous seablite "Schorrekruid" (Suaeda maritima), sea aster or "Zeeaster" (Aster tripolium), Sea arrow grass "Schorrezoutgras" (Triglochin maritima), and perennial sea spurrey "gerande schijnspurrie" (Spergularia media). At higher levels this community is followed by sea purslane "zoutmelde" (Halimione portulacoides). In the final stage of land formation, species like creeping fescue/red fescue "rood zwenkgras" (Festuca rubra littoralis), sea wormwood "zeealsem" (Artemisia maritima) and sea lavender "lamsoor" (Limonium vulgare) together with sea plantain "zeeweegbree" (Plantago maritima) are the most important. More landward of this vegetation one finds the traditional non-halophytic land (read: fresh water) species.

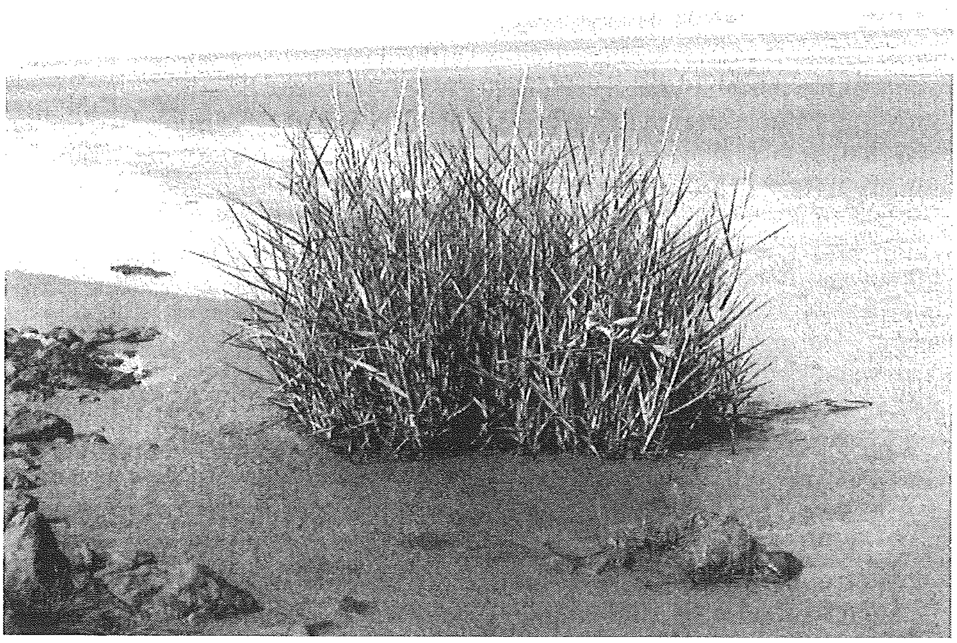


Figure 6-25 Cordgrass (Spartina Anglica) (Packham, 1997)

Due to its dense cover and its ability to grow in a zone extending from 1 m below to 0.15 m above average high water level the pioneer vegetation of cord grass "Engels slijkgras" plays a very dominant role in the accumulation of silt. Its ability to trap sediment, either by physically capturing it from passing currents or by retarding currents and permitting the sediment to settle into the plant community, makes these plants very important contributors to coastal sediment accumulation. In addition to their positive role in catching sediment to the substrate, marsh grasses are very important sediment stabilisers. They prevent or inhibit currents and waves from removing sediment from the vegetated substrate, partly by their root system (strengthening the soil), partly by reducing the current velocities (reducing the load). This grass is therefore often introduced artificially to enhance siltation and formation of new farmland.

Similar sequences of species can be indicated for brackish conditions and for fresh water conditions. When surveying an estuary, one can guess on the basis of the vegetation on the flats and along the shore what the local salinity is.

An extensive marsh is a sign of a natural estuary that has largely filled with sediment. Many marshes mark the locations of old estuaries that have filled with sediment to the requisite elevation for plant germination.

The upper limits of the salt marsh coincide with the landward or upper limit of the spring high tide, the highest level of regular inundation and sediment supply. A salt marsh may be a few meters wide or it may occupy the entire estuary except for the tidal channels. In Figure 6-26, a cross-section of a salt marsh is drawn. This Figure shows the different zones within the marshland that can be distinguished by the different species that are present.

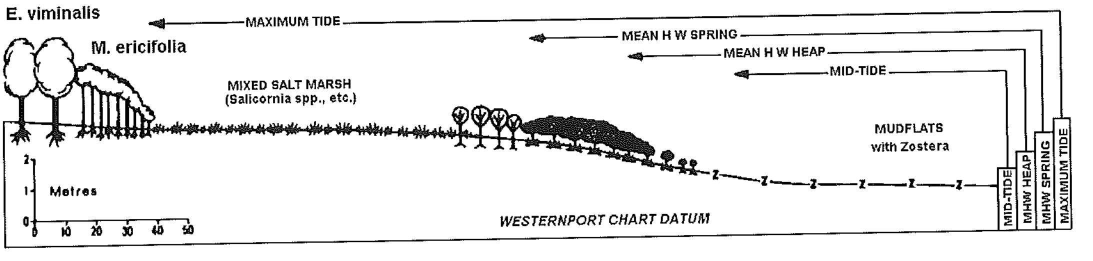


Figure 6-26 Cross-Section of a salt marsh

Individual marsh flats can develop into extremely valuable nature resorts. Apart from many specific vegetation species, animals use the place for breeding, feeding and during seasonal migrations. Beautiful examples of such a marsh in the Netherlands are the Wadden Sea and the tidal flats of Eastern and Western Scheldt.

The marsh environment is quite similar to that of river and delta floodplain. Channels, bordered by natural levees and crevasse splays, cut through the marshy plain. Some channels meander and produce cut-offs and oxbow lakes. This system delivers sediment to the marsh in two ways: regular but slow flooding of the marsh by turbid water carried by sluggish currents that permit settling; and storm tides that push large amounts of sediment-laden water onto the marsh and deposit considerable sediment in a short time. Although a paradigm for marsh development has been given here, the present global situation is one of eroding marshes due to sea level rise.

For hundreds of years, the Dutch, Germans, and Danes have been converting marshes to farmland by draining them through a system of dams, dikes, and canals. This process has now been stopped, mainly because the ecological value of the tidal wetlands has been recognised and the Wadden Sea has been declared a nature reserve.

Whatever the ecological value of the salt marshes, the coastal engineer cannot neglect the fact that the vegetation is an important means to stabilise the fresh sediments and to enhance the natural protection of the hinterland.

6.3.2 Mangrove swamps

In tropical and subtropical climates, extensive stands of mangroves - woody trees of various taxonomic groups - invade the inter-tidal zones of estuaries and other bays, similar to the salt marshes in the moderate zones. They favour silty coasts with a moderate wave climate and a regular supply of clean oxygen-rich water. Thick tangles of shrub and tree roots, commonly called swamps but properly known as mangles, form an almost impenetrable wall at about water level. Most trees grow from 2 to about 8 m high, although some are much higher, depending on the species and the environmental conditions - rare stands may be twice that height.

The root systems of mangroves (Figure 6-27) are not only dense, but also diverse in appearance and function. The most spectacular root display is put on by the red mangrove (Rhizophora mangle), which has large, reddish prop roots that support the tree. It also has vertical drop roots, which are long vertical appendages that sprout from low-lying branches and eventually reach the ground and give support. Pneumatophores, another common type of root, occur in another common species, the black mangrove, Avicennia germinans. The pneumatophore is a short root growing upward from lateral runners extending from the central trunk. Although there is considerable argument about their function, it is believed that they are respiratory organs for exchange of oxygen. The third species of mangrove in Florida and the Caribbean Islands is the white Laguncularia racemosa. In places like tropical Australia or India, there are more than 20 species of mangroves.

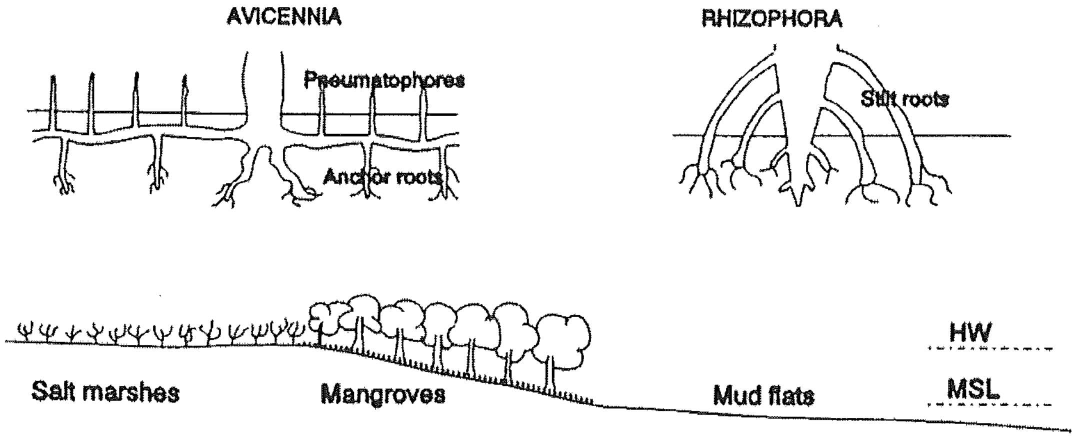


Figure 6-27 Mangrove roots and yypical cross-section of Mangal

The thickets of mangrove roots at the water line provide a sheltered habitat for a special community of organisms that are adapted to an environment intermediate between land and water. Barnacles and oysters encrust the roots and branches, looking almost like fruit. Fish, snails, and snakes all find protection, nesting sites, and food among the roots. Moreover. mangroves provide a practically indestructible coastal defence against storms and hurricanes as long as the conditions are not too rough (Figure 6-28).

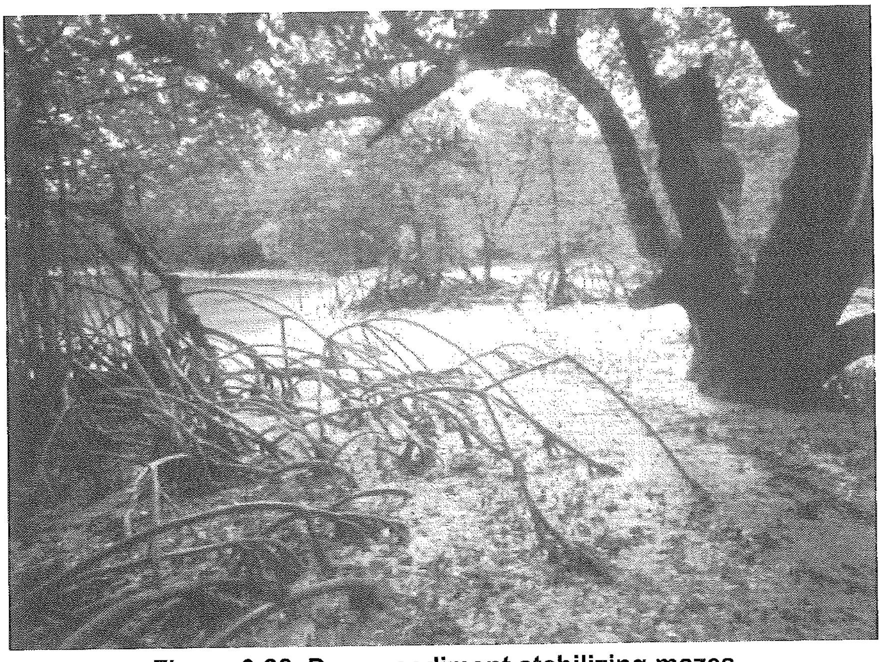


Figure 6-28 Dense sediment stabilizing mazes of the massive Mangrove root systems

The mangrove trees are not only useful in living condition, the local population cuts many of them to provide firewood. Because the mangrove swamps are rich in expensive seafood like shrimp, the shores are often turned into artificial fish and shrimp farms. Destruction of mangrove forests and their replacement by shrimp farms is a major factor responsible for the increase in the severity of flooding in many coastal areas, especially in India and Indonesia. Finally, water pollution is also hampering the continued growth of mangrove forest. This means that the mangrove vegetation is disappearing rapidly. Once the forest has disappeared, it becomes clear how effective the forest was in preventing erosion of the generally silty coastline. Re-afforestation is extremely difficult since the young trees are quite vulnerable.

6.3.3 Dune vegetation

The vegetation discussed in the previous paragraphs requires calm conditions and a silty substratum to germinate. The silty character keeps the soil moist, also during Low Water. The calm conditions and the silty soil are closely related since only in such conditions can the the finer particles settle.

The coarser sand fractions of the sediment (sand) will generally be deposited in a more dynamic environment, such as the deep channels with their high current velocity or the beaches with severe wave action in the breaker zone. This means that even after sedimentation the mobility of the grains remains relatively high, and that any form of life in those locations must be rather mobile as well. It appears that benthic organisms can survive here, but that it is difficult for land plants to colonise such areas.

Only in places where conditions are slightly friendlier for prolonged periods (such as the beach above the HW line), can non-marine plant life survive. Conditions on the dry beach are still quite difficult: the sand has very little capacity to hold any moisture, so that plants growing here must be drought resistant. When the first vegetation develops on the dry beach, it is the nucleus for the formation of dunes. Like the marsh vegetation, the pioneer species "Biestarwegras" (Elytrigia juncea) provides some shelter against the wind where sand can accumulate, while the roots keep the sand together. In this way, the first plants create small slightly elevated undulations in the flat beach. In these higher places, some fresh water can be stored, which creates more favourable conditions for the following species, of which Marram or "Helm" (Ammophila arenaria) is the best known variety. Helm has a more extensive root system and it forms a dense cover with its stems and leaves. In this way, real small dunes are being formed, and the higher the dunes become, the better become the conditions for more varied vegetation. In the sand hills, fresh water is caught and this drains slowly to the lower parts of the slopes where species requiring more water can establish themselves. It is beyond the scope of this textbook to describe all species, but it is worthwhile to visit some of the older dune reservations that have not yet been destroyed by the winning of drinking water, and to admire the richness in vegetation.

Since dunes form an important part of the sea defence, not only in the Netherlands, but also in other parts of the world, the vegetation cover of the dunes is essential because it prevents the "wandering" of dunes, blown by the wind. In the Netherlands Marram or "Helmgras" is used extensively to provide artificial protection to young dunes and to prevent wind erosion. Again, the species are site specific, in the sense that the composition of the soil and the climate play an important role in the survival of the fittest species. This means that the use of vegetation to stabilise sandy shores must always be based on observation of locally available and successful species. In this respect mention is made of a publication dating from the colonial administration in Indonesia, which describes the tropical species prevalent in Indonesia that are suitable for dune stabilisation there (Lieftinck, 1937).

6.3.4 Coral reefs

The term coral reef refers to rigid, sublittoral, structures composed of calcium carbonate. The calcium carbonate is excreted by benthic organisms. The corals (Cnidaria) are one of the dominant types of organisms,; another is Lithothamnion, a coral-like red algae. Many other organisms contribute calcium carbonate to the reef, and their shells and debris are encrusted by Lithothamnion into a well-cemented structure. Some of the most common of those organisms are Halimeda, a green algae, foraminifera, many bivalves, and many gastropods.

Warm water and the penetration of sunlight are essential to the development of coral reefs. Light is important, because Lithothamnion and Halimeda are both photosynthetic plants. Also, since corals are benthic animals that rely on currents to provide oxygen, the seawater must circulate well and be rich in calcium and carbonate ions. Because the warm waters of the tropics are generally deficient in carbonate, the upwelling of relatively carbonate-rich water from depths of 100 to 300 m is necessary for coral reefs to develop. This means, that coral reefs are most likely to form near steep island or continental slopes on the western boundaries of the oceans. The easterly equatorial currents are deflected upward when they meet the slopes, producing an upwelling of nutrient-rich water.

The reef-building corals (Figure 6-29) themselves depend on light penetration. This is because the corals are inhabited by symbiotic, photosynthetic dinoflagellates, called zooxanthellae, which provide oxygen for the corals and remove wastes. The corals in turn provide carbon dioxide, nutrients, and protection for the zooxanthellae, such as radiolaria, sponges, sea anemones, bivalves, and echinoderms. The giant clam, Tridacna gigas, is able to digest zooxanthellae, enabling it to reach a much larger size than it could achieve by feeding on the plankton brought by currents.

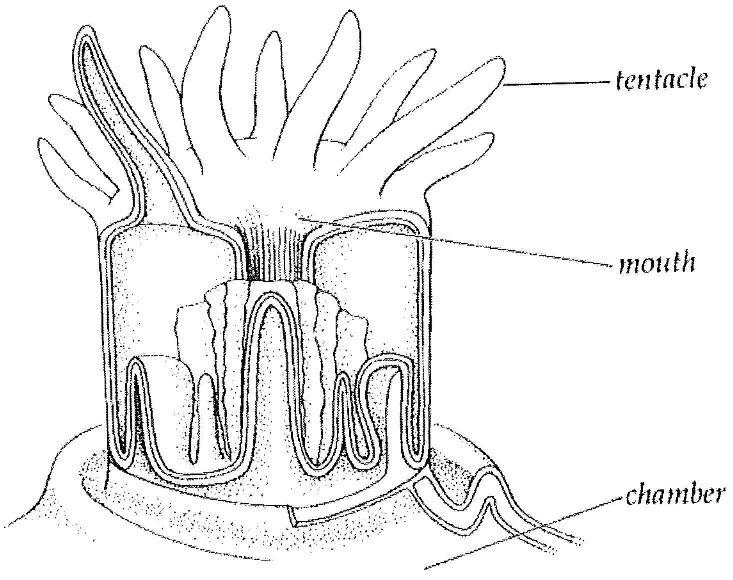


Figure 6-29 Cross-sectional model of an individual coral

The coral reef ecosystem is based on a closed energy cycle. The amount of plankton living at the front of the reef is approximately the same as the amount living at the back. The apparent primary producer of organic nutrients from inorganic nutrients in the system is filamentous green algae, which thrive on the nutrients and carbon dioxide provided by the corals and other animals. Lithothamnion forms an algal rim within the surf zone, and corals and Halimeda inhabit the reef front and reef flat where wave action is less severe. Typhoons and predators such as the crown-of-thorns starfish (Acanthaster) can decimate corals. When this occurs, Lithothamnion or other encrusting algae form on the coral skeletons.

Coral reefs are, like tropical rain forests, among the most complex communities on earth, and rock-producing reef communities are among the most ancient life forms found in the fossil record. Because of their complexity, the dynamics of coral reefs are not yet well understood. During the most recent centuries coral reefs have been adversely affected by humans. Some of the most widespread impacts are water pollution from various human activities, deforestation (erosion leading to increased turbidity), dredge and fill operations, over-harvesting of fish and shellfish, and the harvesting of some corals for souvenirs. All forms of stress on the coral retard its growth. Destruction cannot be replaced; natural recovery would take thousands of years. Implantation experiments have been carried out, but they have failed. This is serious, since many low-income communities depend on the coral reefs to protect their property against flooding by high tides and wind set-up.

Stoddard (1969) has identified four major forms of large-scale coral reef types (Figure 6-30):

1 Fringing reefs
2 Barrier reefs
3 Platform reefs
4 Atolls

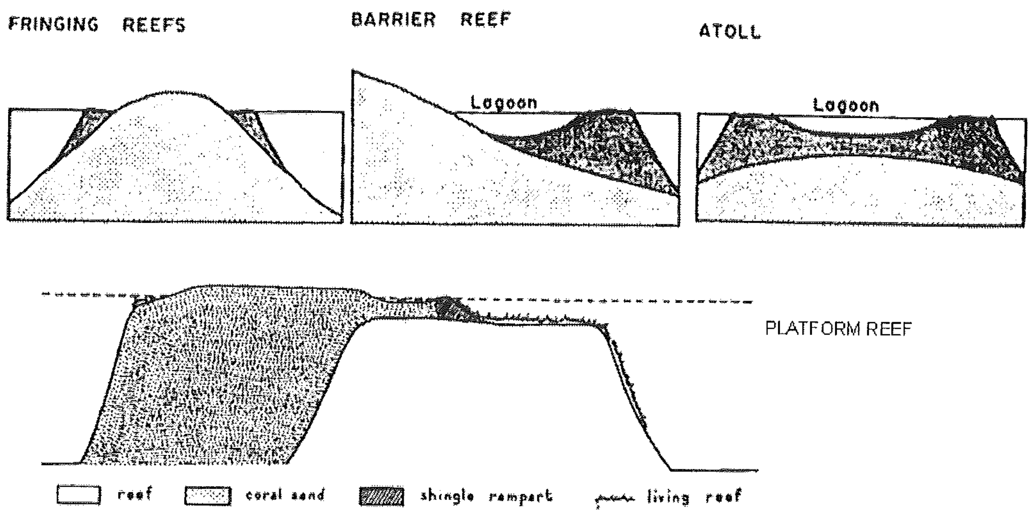


Figure 6-30 Reef landform types (Bird, 1983 and Verstappen, 1953)

Where reefs border the coast they are termed fringing reefs; where they lie offshore, enclosing a lagoon, they are known as barrier reefs; and where they encircle a lagoon, they are called atolls. Platform or table reefs form on shallow banks that have been capped with reef-forming organisms. They cover extensive areas of the seafloor and are not associated with the formation of barriers and lagoons.

Atolls are ring-shaped reefs that grow around the edges of extinct volcanic islands, enclosing lagoons. The shallow lagoons may contain patch reefs. Atolls are primarily found in isolated groups in the western Pacific Ocean. Small low islands composed of coral sand may form on these reefs. These islands are quite vulnerable to inundation, and to tropical storms. The first theory concerning the development of atolls, the subsidence theory proposed by Charles Darwin in 1842, has been shown to be basically correct (Strahler 1971). The evolution of a coral island exists in different phases, as drawn in Figure 6-31:

a Active volcano rising from the seafloor
b Extinct volcanic island with fringing reef
c Subsiding island; reef builds upward and seaward, forming barrier reef
d Continued subsidence causing remnant volcanic island to be completely submerged

Growth continues upward and seaward until the remnant volcano is covered.
It is important to stress that reef islands are naturally dynamic. Sediment production occurs around reef islands, and erosion, deposition and cementation can occur concurrently on atolls today (Wiens, 1962). Some islands may be in a stable equilibrium with neither addition nor loss of sediment. However, on most islands, sediment is added and lost over time and there is more likely to be a dynamic equilibrium between inputs and outputs. Islands adjust over a range of time scales.

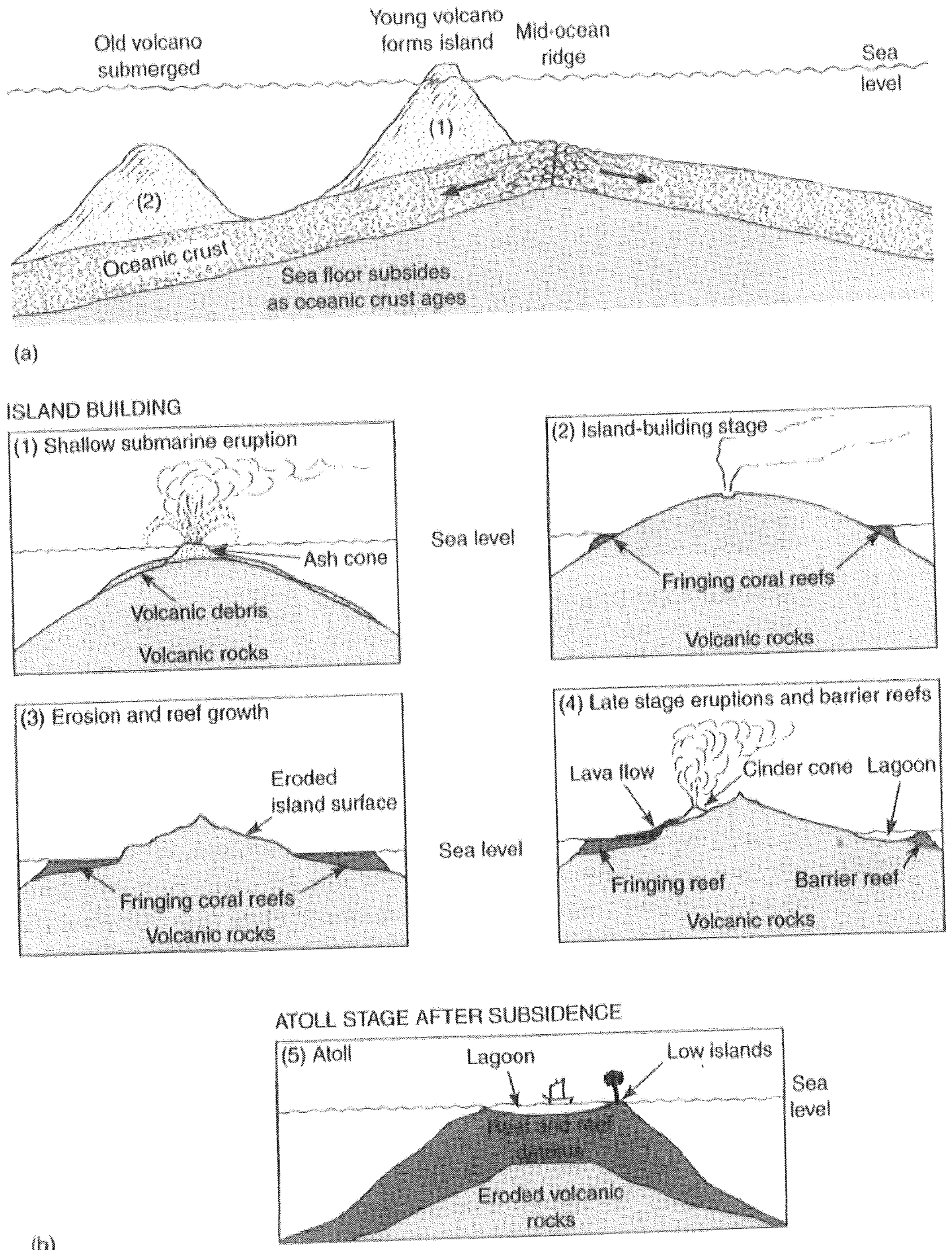


Figure 6-31 Evolution of a coral island (adapted from Press and Siever, 1986)

6.4 Rocky coasts

6.4.1 Origin of rocky coasts

The largest part (75 %) of the world's continental and island margins consists of rock. Rocky coasts are commonly tectonically active convergent coasts, which produce a high-relief border. Because they are formed on continental plate margins, under which an oceanic plate is descending, virtually no continental shelf is present. The western edges of North and South America are excellent examples of this type of coast. On other, tectonically unrelated, cliffed coasts, various sedimentary strata are horizontal or dip at low angles. The adjacent continental shelf is wide, with a gentle slope.

Pleistocene glaciers have also had a hand in producing cliffed coasts. The moving ice masses gouged out steep valleys, which were subsequently drowned as the sea level rose. Although their profiles are similar, some are rocky and others are not. In Figure 6-32, the rocky coast along a fjord at Kenai Fjords National Park, Alaska, is shown. It was carved by a glacier.

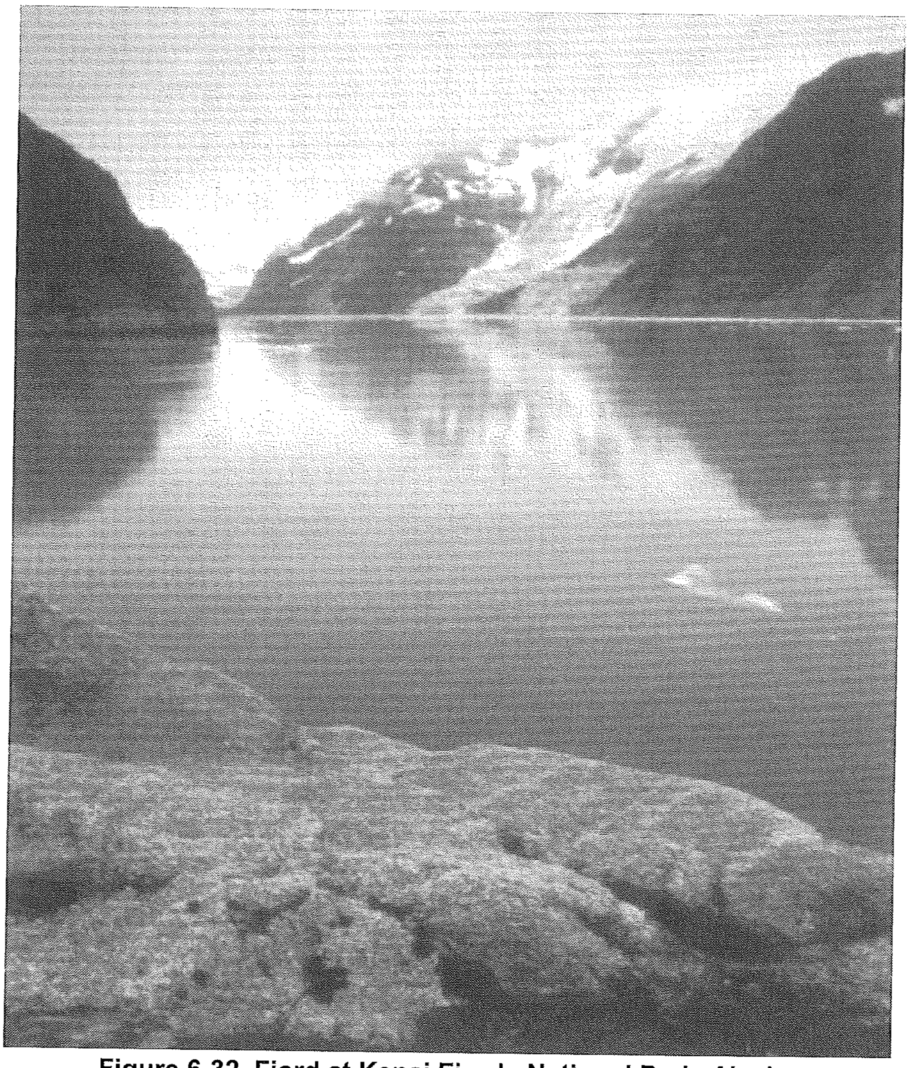


Figure 6-32 Fjord at Kenai Fjords National Park, Alaska

Still other cliffed coasts are formed of glacial till, sediment deposited by glaciers beneath and at the margins of the ice. The till is over 100 m thick in some places and includes nearly any type of material from stiff clays to sand, gravel and boulders. Some of it is well layered and some is massive, with essentially no internal coherence. The accumulations known as end moraines tend to be linear and thick. When these end moraines meet the sea, the waves sculpt steep bluffs. Irregular bluffs of glacial drift show erosion on the coast at Gay Head, Martha's Vineyard, Massachusetts, Figure 6-33.

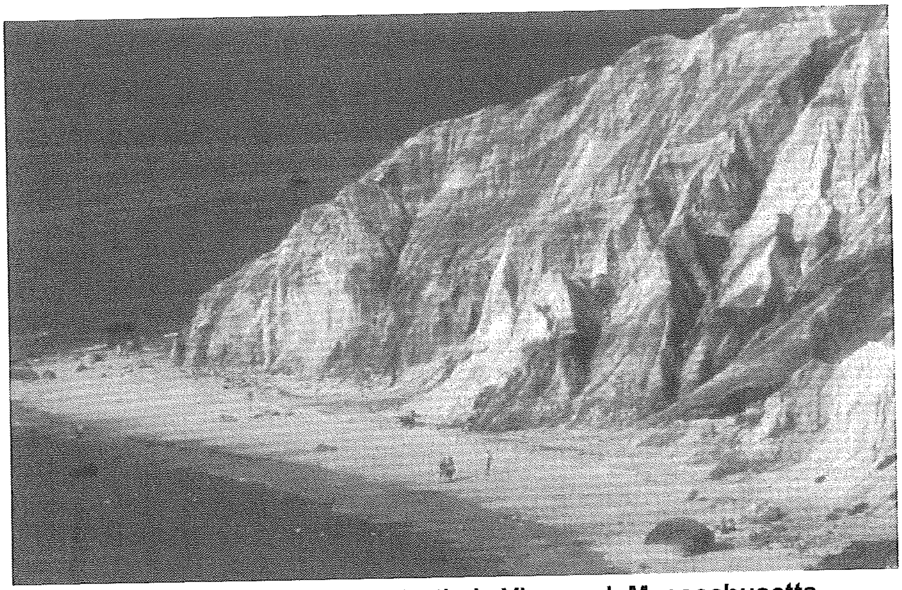


Figure 6-33 Gay Head, Martha's Vineyard, Massachusetts

Another variety of rocky and commonly cliffed coast is associated with areas where the continental shelf and adjacent coast are dominated by skeletal shell debris. A similar type of rocky coast has been constructed from the abundant carbonate sediment in these contrasting areas. In the Pleistocene, onshore winds blew carbonate sediment landward, where it accumulated in wide beaches and dunes. In a process called lithification, the calcium carbonate grains are welded together by a cement created as ocean spray or percolating ground water reacts with the calcium carbonate. The evaporation of the regularly wetted surfaces in arid climates enhances the lithification of the sediments. The rapid cementation converts the dunes to a rock called eolianite. The coast of N. Africa is well known for its cemented sand, which hampers dredging operations because in surveys it appears to be sand, whereas it is quite hard in reality..

6.4.2 Rock erosion

Although rock is considered to be a strong material, rocky coasts can still be subject to erosion. Along rocky coasts, nearshore wave energy is often high because the size of the waves is related to the nearshore bathymetry and refraction patterns. The wave energy is focused on the headlands and dispersed in the bays, so the headlands erode as the intervening bays fill up. Wave erosion of an indented coastline produces a straightened, cliff-bound coast, as shown in Figure 6-34. Wave-cut platforms and isolated stacks and arches may be left offshore.

Wave impacts of breaking waves may also cause erosion through the pressure wave that is hitting the rock and propagating through voids and fissures. As soon as material breaks off, the larger particles move across the rock surface and produce considerable impact and abrasion (Figure 6-35).

Waves, however are not the only phenomena that cause erosion of rocky coasts. Other, subtler physical processes contribute to change rocky coasts. Evaporation and temperature change can cause mineral grains and rock fragments to expand and contract slightly. When water freezes under confinement, it can break rocks. Porous and permeable rocks are particularly vulnerable to frost damage. In tropical areas heating and cooling cause huge stresses in the material that will also cause erosion.

However the rate of erosion of rocky coast is so slow, that remedial measures are seldom required.

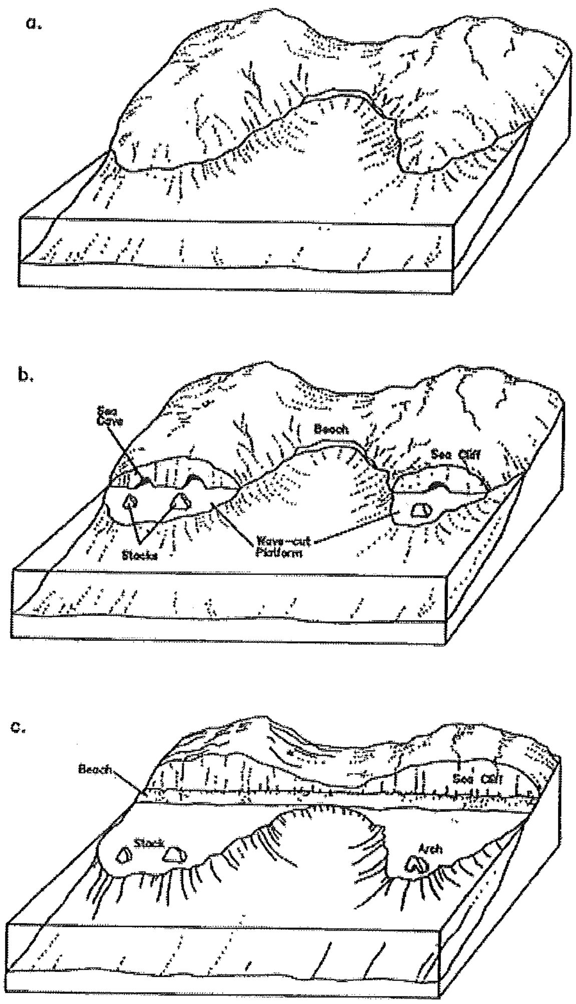


Figure 6-34 Wave-erosion Effects (adapted from De Blij and Muller, 1993)

Figure 6-35 Rock perforated by spherical hollows, Called Tafoni, San Mateo County, California

7. COASTAL ZONE MANAGEMENT

7.1 Introduction

In the previous chapters, it has been indicated that to give an exact definition of the coast or the coastal zone is rather complicated. Processes that contribute to the physical conditions of the coast have been explained and it has become evident that the coastal zone extends considerably seaward and landward of the coastline which forms the actual boundary between land and sea. The physical processes were largely connected to the geological history of the area and to the sediment transport that continuously reshapes the coast. Thus, whatever definition we give of its extent, the coastal zone is a very dynamic part of the earth's crust,

The dynamic nature of the coastal zone is not confined to the physical properties. Owing to the varying physical conditions and the large gradients in the physical conditions, the coastal zone also provides habitats for an extremely wide range of flora and fauna. Therefore, the coastal zone probably makes a larger contribution to the bio-diversity than any other specific region on earth.

Throughout recorded history, and probably long before that, the sea has been a recurrent theme in human culture: in religious accounts, in folklore, and in scientific investigations. Archaeologists confirm that human societies have long had close ties to the sea and its shores, ties that persist to this very day. This is not a surprise. The coastal zone provided for almost all human needs such as drinking water, fertile land, ample resources in terms of game, (later domesticated cattle) and fish, and even a sink for the waste products of the human society. In later stages, the sea provided opportunities for transport facilities and strategic protection against enemies. Thus, the coastline became a very attractive zone to live, to work and to relax.

This condition, however, is changing rapidly. Because of the unique character of the coastal zone, it is necessary to consider these changes in more detail.

The coastal zone can be seen as the playing field for many social and economic developments and resulting conflicts of interest. Coastal engineering itself plays only a limited role since the boundary conditions for any works seem to be dominated by economic, political and legal considerations. On one hand the coastal engineer must therefore have a good understanding of the global changes and on the other hand be familiar with the socio-economic subsystem. Moreover he must accept and understand the role of the controlling bodies in society. Politicians and decision-makers in their turn must be aware of the fact that nature imposes boundary conditions that cannot be changed by party programs or elections. Many of these boundary conditions have been discussed in the previous Chapters, and it therefore does not seem necessary to pay more attention to the natural subsystem in this Chapter.

7.2 Global changes

7.2.1 Growth of the world population

With the growing world population, an ever-increasing number of people inhabit the coastal zone. This applies not only to absolute numbers, but also to the percentage of the world population (Figure 7-1). They concentrate in extremely large cities, as is made clear by Table 7-1 and Figure 7-2. Most of these large urban communities are located in the coastal zone.

City	Country	Population in million persons		
		1950	1995	2015 estimate
Tokyo	Japan	6.92	26.96	29
Mexico City	Mexico	2.88	16.56	19
Sao Paulo	Brazil	2.42	16.53	20
New York	USA	12.34	16.33	18
Bombay	India	2.9	15.14	26
Shanghai	China	5.33	13.58	18
Los Angeles	USA	4.05	12.41	14
Calcutta	India	4.45	11.92	17
Buenos Aires	Argentina	5.04	11.8	14
Seoul	Korea	1.02	11.61	13
Beijing	China	3.91	11.3	16
Osaka	Japan	4.15	10.61	11
Lagos	Nigeria	0.29	10.29	25
Rio de Janeiro	Brazil	2.86	10.18	12
Delhi	India	1.39	9.95	17
Karachi	Pakistan	1.03	9.77	19
Cairo	Egypt	2.41	9.69	14
Paris	France	5.44	9.52	10
Tianjin	China	2.37	9.42	14
Moscow	Russia	5.36	9.3	9
Manila	Philippines	1.54	9.29	15
Jakarta	Indonesia	1.45	8.62	14
Dacca	Bangladesh	0.42	8.55	19
London	UK	8.73	7.64	8

Table 7-1 Urban population

Inland mega cities shaded in grey

All the various functions of the coastal zone result in a shortage of space, and ever more different functions of the coastal zone compete for the same scarce area. This means that a larger proportion of the coastal zone is being used by human society. It also means that areas are being occupied that pose serious threats to their users because of their location close to the actual water line and the associated risk of flooding. The dynamic morphological behaviour of the coastline becomes a serious concern. The growing pressure on the coastal zone also leads to conflicting use of space, since not all designated functions can be combined. This could perhaps be solved by the enforcement of strict zoning regulations, but such regulations would first have to be established. That would not be easy since all concerned naturally try to influence the legislators to give preference to their specific interest.

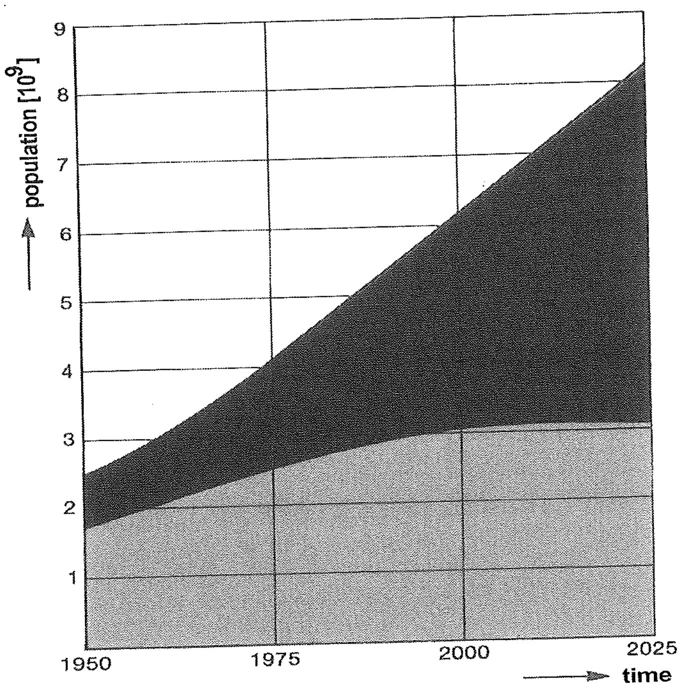


Figure 7-1 Development of world population
Black indicates coastal population

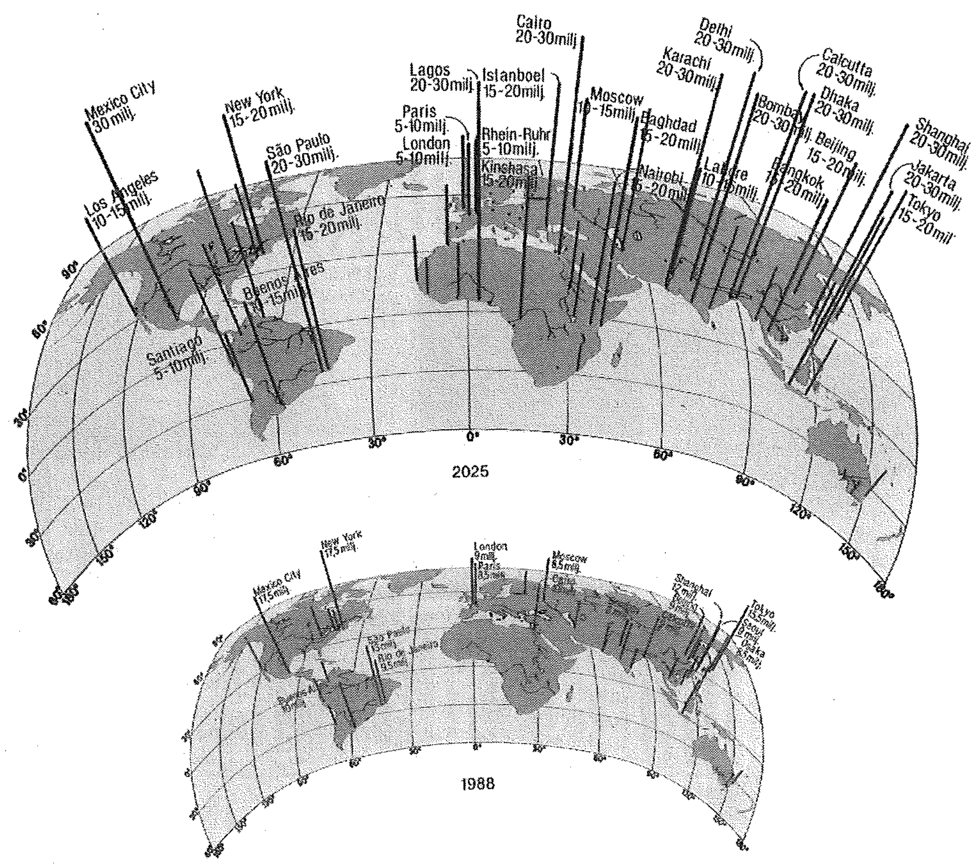


Figure 7-2 Development of urban conglomerations in the world

It also appears however, that the natural resources of the coastal zone are not sufficient to cope with the growing demand. The rich resources of the coastal zone are being rapidly depleted which endangers the sustainability of the unique ecosystem, both on land and in the water. In order to safeguard the sustainability of the coastal zone the only remedy is to include the interests of the ecosystem in the spatial planning considerations. This is coastal zone management. Sustainability is defined as the possibility for future generations to use the resources to the same extent as we have been using them to date.

7.2.2 Climate change and sea level rise

On a geological time scale, changes of climate are not a rare phenomenon. They have occurred locally when the plates moved across the earth and passed through different climate zones. They also have occurred globally for reasons that we do not know. Large volcanic eruptions or the impact of meteorites may have played a role. We have evidence of the occurrence of glacial and interglacial periods in recent geological history. It is therefore unlikely that the present climatic conditions and the present sea level remain the same forever. On the contrary, it must be expected that changes will take place, and based on observation of the recent geological history, these changes may be quite large and quite rapid. Any change in global temperature will have an impact on the global sea level, and we must realise that the sea level changes during the most recent centuries have been quite moderate. This has certainly contributed to the popularity of the coastal zone for the purposes of living, working and recreation.

Due to the extensive use of fossil energy sources by human society during the last century, we have added a man-made element to the system. The emission of large quantities of carbon dioxide into the atmosphere, causing changes in the insulating properties of the atmosphere, and it is expected that this will result in a gradual rise of the global temperature. This is often referred to as the greenhouse effect. Calculations are being made to estimate rise in temperature and what the effects of this will be on the world climate. The models used for these calculations all have a rather tentative character since we do not know all parameters that play a role and therefore it is impossible to validate the models. The period of observation is too short for them to have been calibrated either. Nevertheless, it must be expected that the greenhouse effect will at least have some influence on global temperatures and the global climate. It is widely expected that one of the consequences will be acceleration in the rise of the sea level, due to melting of the ices in polar regions. There will also be changes in storm and rainfall patterns. The actual rate of sea level change is difficult to predict, but in the Netherlands, it is considered that a value of 0.5m during the 21st century is likely. If this should be the trend all over the world, some densely populated areas will face serious problems.

The global warming due to the greenhouse effect is not the only element that contributes to a rise in sea levels. In many coastal areas, ground water is extracted on a large scale, partly to provide drinking water and, partly to improve drainage for agricultural purposes. When the lowering of the ground water table takes place in a region with compressible subsoil, the area will settle and the land will be more vulnerable to flooding. Examples of such settlement can be found in the Netherlands (Figure 7-3), in Thailand around Bangkok and in Indonesia around Jakarta.

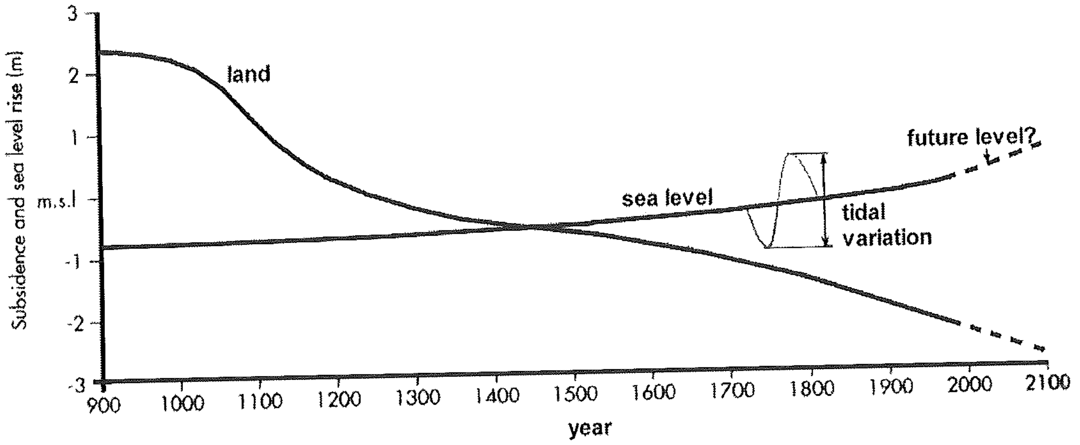


Figure 7-3 Ground level drop and sea level rise in the Netherlands

When estuaries and deltas are concerned, there may be a combination of all factors: sea level rise, settlement of the land, increased river discharges and an increase in storm surges due to climate changes. These compound effects definitely contribute to the pressure on the coastal zone.

One of the problems in counteracting the greenhouse effect is the international nature of the emission of carbon dioxide. It makes little sense for an individual country to reduce the emission. Such decision would reduce the standard of living there in comparison to that of neighbouring countries that continue emissions at the same level, and it would scarcely contribute to a solution of the global problem.

7.2.3 Pollution

In fact, the emission of greenhouse gases is a particular aspect of a more general problem: the discharge of by-products and wastes from our industrial production processes into the environment. As early as the middle of the 1970's, it was recognised that industrial wastes such as heavy metals, polychlorinated hydrocarbons and nuclear waste material were being discharged into open water (rivers and seas) on a large scale

Specifically the heavy metals and the chlorinated hydrocarbons were bonded onto the clay particles suspended in the water by electrochemical forces. In this way, the pollutants spread with the suspended material rather than with the water. The pollutants accumulated in areas where the fine sediments settled and from there, they found their way in the food chain. Places that are traditionally the settling basins for the fine sediments, i.e. the flood plains of rivers, the delta areas and the estuaries were seriously affected. The problems surfaced first in Japan, where isolated communities living on a diet that was heavily contaminated, showed signs of new diseases (Itai-itai and Minimata disease). These diseases were later linked to an excessive presence of heavy metals and pesticides in the staple food (fish and rice).

The pressure of several NGOs, focussed international attention on this problem and the first international treaties were signed by the countries bordering the North Sea to limit the practice of waste discharge in open water. Gradually, the working areas of the treaties have been extended, both in geographical sense and in the categorisation of wastes and in the definition of countermeasures and acceptable practices. The port authorities were one of the groups that were heavily hit. The sediments that were traditionally dredged from the ports for maintenance of the fairways and port basins and dumped into open water or on land (for soil improvement) were categorised as chemical wastes. Disposal in open water was no longer permitted and soon restrictions were imposed on disposal on land as well. The ports felt victimised because they were made responsible for solving a problem that was essentially caused by others (the chemical industry) outside the jurisdiction of the ports, often even in other states or countries.

By now the stipulations of international treaties have been incorporated into national legislation in many countries (in the Netherlands the WVO, Wet Verontreiniging Oppervlaktewater 'Act on the contamination of surface waters'). This has gradually put an end to the practice of discharging wastes in open water but it has not solved the problem of the contaminated sediments. In many locations in the coastal zone, the sediments are still contaminated. Most Port Authorities in W. Europe and the USA have developed dredging and disposal methods for the sediments that they have to dredge. It must be expected, however, that special cleaning operations are still required for areas that are not dredged regularly for the purpose of navigation. In the Netherlands, the disposal areas "Slufter" near the Port of Rotterdam and "IJsseloog" in the mouth of the river IJssel (tributary of the Rhine) are examples of costly measures that had to be taken to be able to continue dredging of essential connections. These disposal areas are meant to prevent a further uncontrolled dispersion of the contaminated dredged material. Real immobilisation of the chemicals by thermal processes is too expensive at this stage.

Further international co-operation will be required to solve the problem of trans-boundary pollution. Special attention is required for the developing countries where the funds are lacking to take restrictive measures at the sources of pollution, and where pollution of the coastal zone can easily lead to diseases similar to those found in Japan.

7.3 The socio-economic subsystem

Social and economic activities are found everywhere in the coastal zone. The coastal zone can be described in socio-economic terms. The central issue here is the interest in quality of life. Human wellbeing depends directly or indirectly on the environmental conditions in the broadest sense.

Any coastal zone usually has many different functions, which are all relevant for human wellbeing. Which functions are most significant depends on the ecological characteristics, the socio-economic circumstances and the management objectives or political priorities of the area in question. The functions are listed in Table 7-2.

Distinctions are made between four main categories of use and within each main category there is a multitude of functions. The complexity of the problem is already demonstrated by the fact that the same functions appear in different main categories. Fisheries for instance can contribute to the low-cost local food supplies. On a larger scale, it can be helpful in providing employment and other economic benefits to the region. The generation of economic benefits can become a source of conflict when the local supply of food is endangered.

These functions do not even include the need to safeguard the environment in terms of sustainability or bio-diversity or refer to values like cultural heritage or landscape.

Main categories	Functions within the main category	Examples	Potential consequences
Basic	Food	Agriculture Fisheries	Eutrofication by fertilisers Depletion of resources
	Water supply	Drinking water Irrigation	Depletion of aquifers Salt intrusion via rivers
	Energy	Power plants	Air pollution Cooling water problems
Social	Housing	Residential quarters	Loss of valuable land Loss of landscape values Risk of flooding
	Recreation	Local: restaurants, pus, theatres, etc Sports facilities	Space Noise Hooliganism
Economic	Transport	Ports and Harbours Airports	Space Pollution of air and water Noise Effects of dredging Erosion
	Mining	Extraction of minerals like oil, gas, salt, etc.	Noise Pollution Subsidence of land
	Industry	Factories	Pollution Noise
	Agriculture	cattle breeding Fruit plantations	Eutrofication Plant diseases Excess of manure
	Aquaculture	Shrimp farms	Erosion Diseases
	Fisheries	Fishing on the high seas Canning Freezing	Depletion of local resources
	Recreation	Hotels Camping sites Marina's nature reserves	Space requirements Noise Undesirable activities
Public	Mobility	Roads Railroads Cables and pipelines	Space requirements Noise Pollution Accidents
	Defence	Naval base Shooting ranges	Space Noise
	Sewage	Sewer system Treating plant	Water Pollution Space, smell
	solid waste	Incineration dumping (marine or land)	Air pollution Pollution. Loss of landscape values

Table 7-2 Use of the coastal zone and potential hazards

Each activity can conflict with conditions required for any of the other activities. That means that any integrated approach must start with an inventory of the actual functions, their spatial requirements and their characteristics. The most sensible way to do this is to make use of Geographical Information Systems (GIS). They provide a basis for further analysis.

To obtain an idea of the complexity of the use of the coastal zone, the reader can refer to Figure 7-4, which gives an inventory of functions in the area where the Netherlands offshore airport was envisaged.

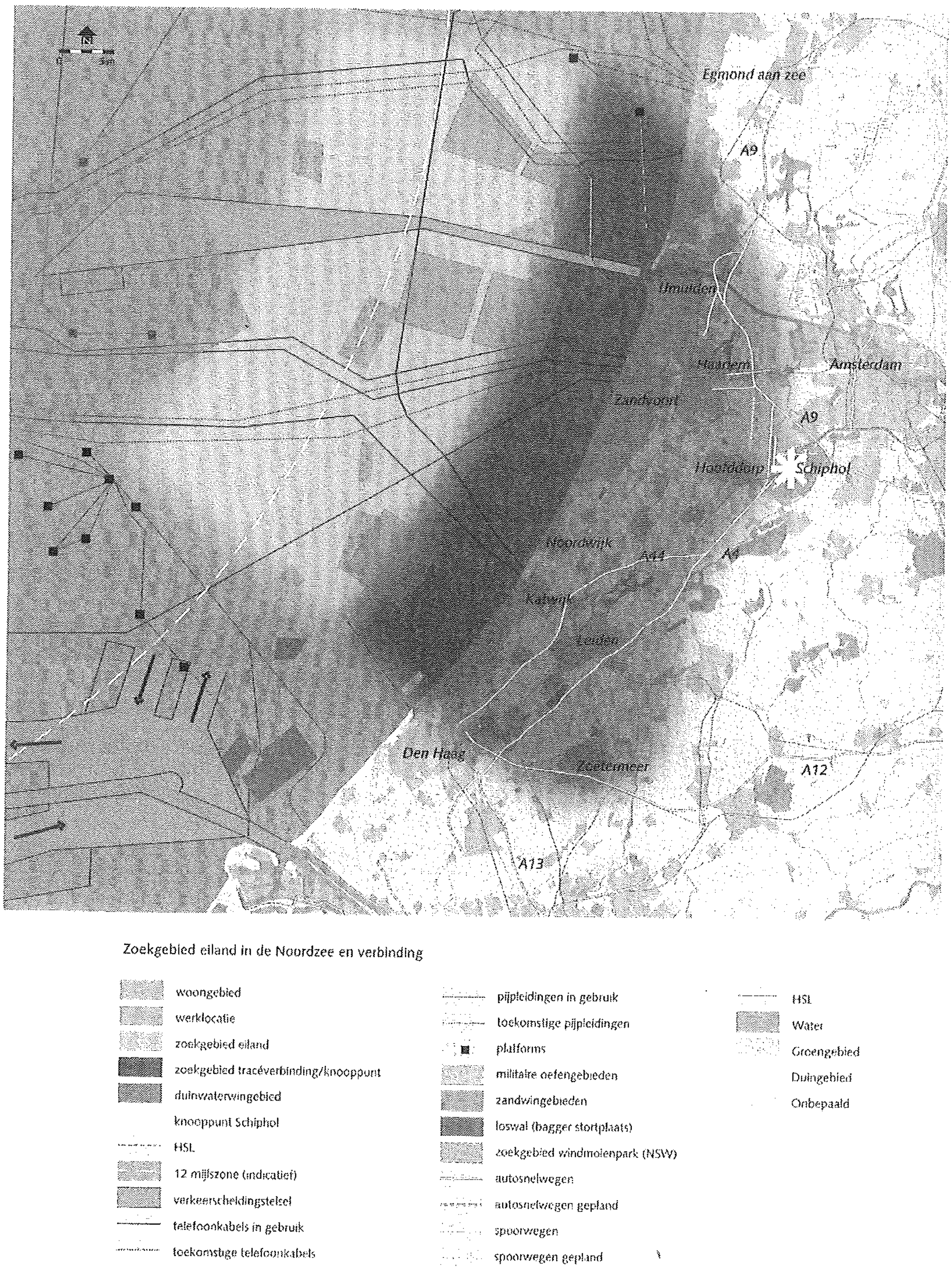


Figure 7-4 Functions in the area of the envisioned Netherlands offshore airport

7.4 The necessity of management

Historically, the major functions of the coast were limited in number, in complexity and in size. If there was anything to manage, this management was purely an engineering task (construction and maintenance of the facilities and the coastline). Recently, other priorities have developed including natural values, water quality and recreation. The pressure on the coast has increased and our priorities have changed. Kamphuis (1997) summarises these pressures and priorities in Table 7-3 and Table 7-4.

Population Density	• Historically, population densities were high along the coasts • 50% of the population of the United States lives near the coast • 80% of the population of Australia lives near the coast • >80% of the population of Canada lives near its oceans or the Great Lakes • Most of the world's major cities are near the coast
Recent Migration	• Younger, more affluent people value the life style projected by coastal areas • Redevelopment of coastal areas and high real estate values result in high-density development of living space • Many people can now afford to live near the coast in spite of high real estate values • Increased income levels enable people to purchase and use recreational equipment that was previously unthinkable, like yachts, jet skis, parasails, etc
Tourism	• People can afford to go on vacations to far away places and often choose a coastal area • There has been a tremendous increase in air traffic, particularly of package vacations at destination resorts
Linear	• The coast is a always narrow, linear strip of land which receives visitors from cities, states, etc. whose population density is measured as a function of area • If a new "coastal area" is developed, the focus is still always on the coastal strip
Erosion	• Most of the world's coasts are eroding, partly due to global sea level rise

Table 7-3 Pressures on the coast (Kamphuis, 1997)

Initially, people who lived near the coast were closely involved with it. They were fishermen, sailors, dock labour, traders or workers in the factories that existed along the coast. They also lived in a tenuous balance with the coastal resources. The recent large migration to the coast has resulted in stress and overloaded conditions. Many coastal zones have become economically dependent on tourism and recreation. Where the knowledge-intensive, technological industry has been developed, this has also caused an influx of urban professionals to the places where the standards of living are highest (along the coast). With this different population, different values became important in the local society, the changing attitude is clearly demonstrated in Table 7-4.

An important constraint on the coastal zone is that it is essentially linear; it is a narrow strip of land (a few kilometres wide) along the coast. This puts high pressure on land prices and recreational facilities. The coastal zone is essentially a very scarce commodity. Finally, the coastal zone is very fragile, and there is a worldwide tendency for coastal formations to erode. This puts high priority on protecting and maintaining what is there, particularly because real estate values along the coast are so high.

Higher Priority	Lower Priority	Changed Priority
Residential	Industrial and Commercial	Fishing
Recreational	Agriculture	Waste Disposal
Nature Reserves	Transportation	
Aquaculture	Military and Strategic	

Table 7-4 Changes in priorities as conforming use (Kamphuis, 1997)

To cope with these problems, methods have to be developed to analyse them, to make decisions and to enforce decisions through legislation. It is good to realise that problems can be addressed in two different ways. One is the scientific way: combine facts and figures on the basis of knowledge and come to conclusions. The second one is the administrative or political way: neglect the knowledge path, make a choice on the basis of goals and means and come to a decision. It is evident that neither of these simplified methods leads to satisfactory results. In a balanced decision making process, all these aspects find a place. This is illustrated in Figure 7-7 through Figure 7-9.

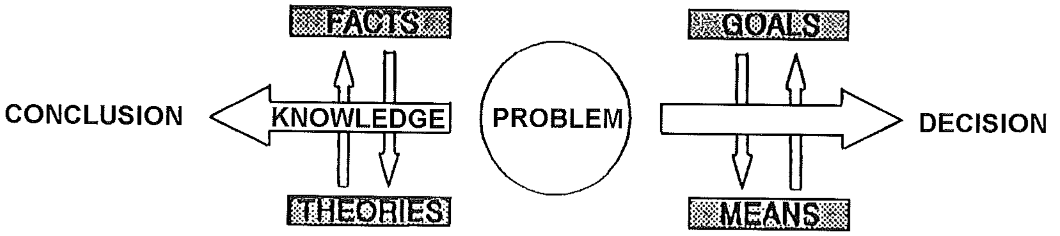


Figure 7-5 Paths of knowledge and choice in decision making (Bos, 1974)

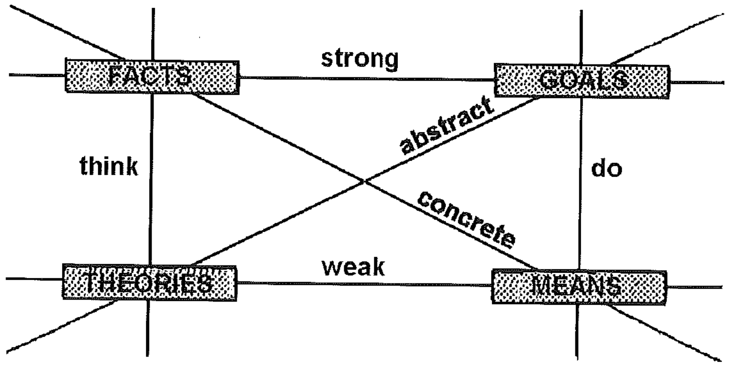


Figure 7-6 Short-cuts in decision making

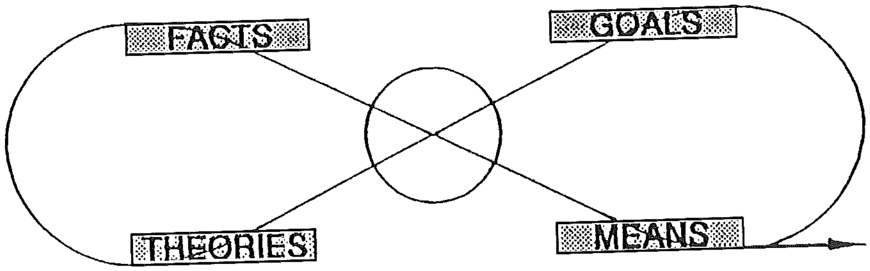


Figure 7-7 Well-balanced decision process

The purpose of exercising Coastal Zone Management is to bring the socio-economic subsystem into balance with the natural subsystem. The structure of the systems and their relation with user functions and infrastructure is sketched in Figure 7-8.

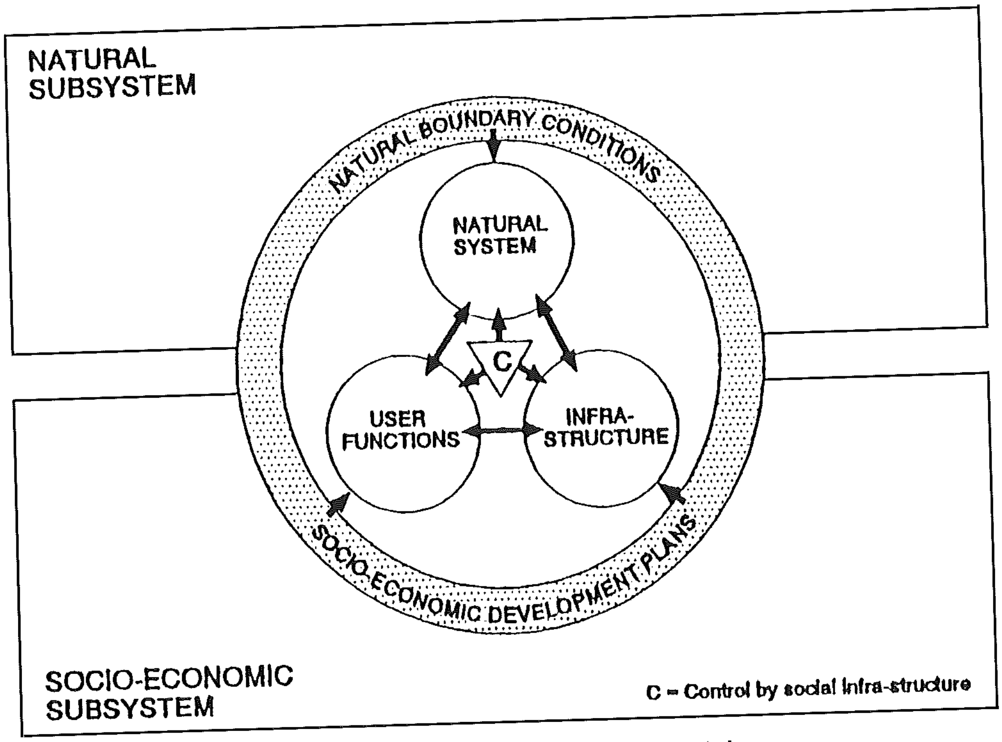


Figure 7-8 A systems view of the coastal zone

Moving in such a labyrinthine system requires a logical, systematic stepwise approach. A logical sequence for such stepwise approach is indicated in Figure 7-9.

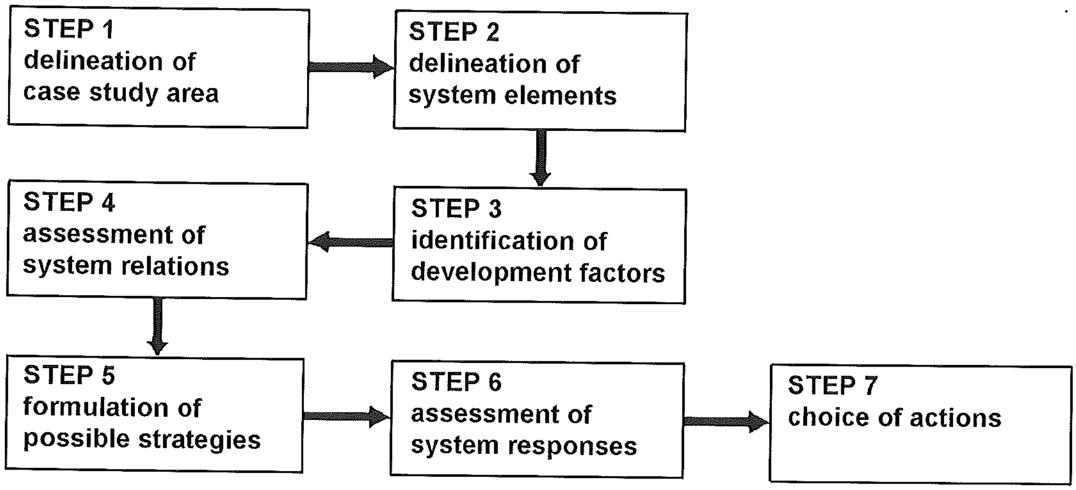


Figure 7-9 Stepwise approach of CZM problem

The steps comprise of the following:

1. Delineation of case study area

The limits of the area to be studied must be determined, both geographically and socio-economically. This is the outer circle in the system diagram (Figure 7-8). The relevant factors from the subsystems are described from available field data and macro-economic data.

2. Delineation of system elements

Databases for the elements within the area to be studied are described from available or newly derived material. These are the inner circles in the system diagram.

3. Identification of development factors (scenarios)

An inventory is made of relevant processes and plans, of both the natural and the socio-economic subsystems. These are the arrows from the outer circle in the system diagram to the system elements (inner circles). They can be seen as the agents of change in the system elements. These agents can be either demand driven (from the socio-economic subsystem) or driven by natural processes.

4. Assessment of system relations

A model is made of the relations between the various elements of the system. In this model, the effect of changes in one system element on the other elements is described. This can be done in a matrix of possible conflicts of interest, describing qualitatively the possible effects. These effects are used in the next step, which is to design promising strategies.

5. Formulation of possible strategies

With the information gathered in the previous steps, it is now possible to design strategies that look promising or that are advocated by some interest group. This is where the CZM control centre, the triangle in the centre of the system diagram, comes into the picture. This can be an administrative institution or combination of involved interest groups. The decision does not reflect the interests of a particular group. The CZM control centre operates at a level beyond the scope of any single interest group.

6. Assessment of system responses

In the system diagram, these are the same arrows as in step 4, but now the effects are quantified for the particular strategies that were developed in step 5.

7 Choice of actions
In the final step, the control centre takes care that a decision is taken by the appropriate authorities on the preferred actions.

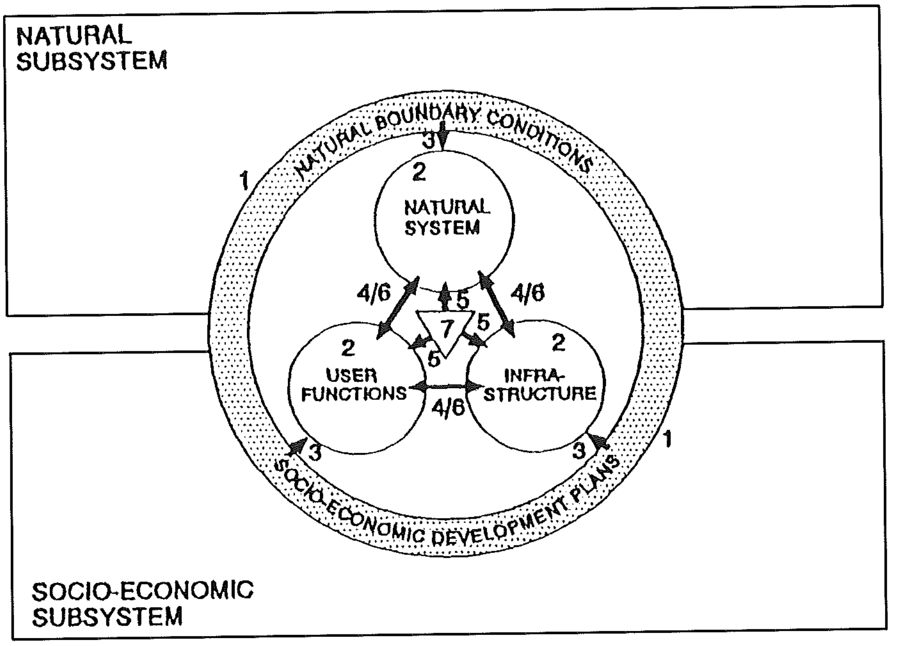


Figure 7-10 Steps related to system diagram

This stepwise approach ensures that the elements facts, theory, goals and means from the decision making process described by Bos are used properly. (Figure 7-11)

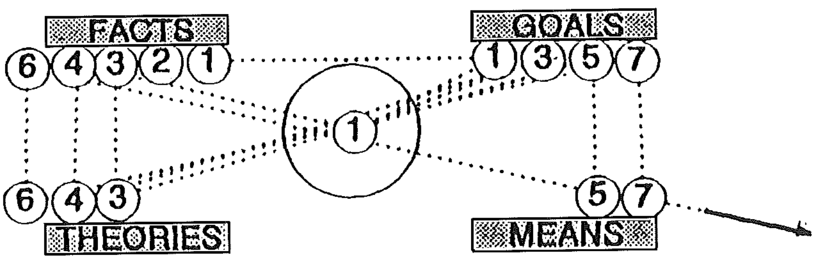


Figure 7-11 Alternating in the Bos diagram by means of a stepwise approach

7.5 Management tools

7.5.1 Weighing the interests

From the above it is clear that CZM is a continuous decision-making process. The problem formulation, the formulation of management objectives and the design of appropriate policies should follow a systematic procedure of generating, analysing and evaluating alternative strategies.
Policy analysis is centred on the comparison between the future and the desired situation. Many aspects of the future and desired situation must be taken into account, especially the different interests of all the parties that are involved. One of the problems is that the interests of these parties cannot always be expressed in the same units. Compare, for example, purely economic

interests that can be calculated in money with the interests of the environment or the number of victims drowned during floods, which have a largely emotional value. Advantages of policy analysis are evident in the following situations:

- When social issues are involved
- When there are many contradictory interests
- When non-comparable values are to be judged
- When there are many values to be compared.

The use of policy analysis becomes more complicated in the execution phase. In many cases, iteration loops have to be made. Goals and standards must be written down at an early stage. There are two kinds of questions that should be submitted to a policy analysis:

- Is it necessary to carry out a project?
- Which alternative is the best one?

One basic management tool is the compatibility matrix. Examples may be found in Carter (1988). Kamphuis (1997) gives a specific example for the coastal zone. Each of these conflicting interests incorporates its own set of conflicts.

		a	b	c	d	e	f	g	h	i
a	Residential	x								
b	Recreational	-1	x							
c	Nature Reserves	-1	2	X						
d	Aquaculture	-1	-2	-1	x					
e	Fishing	1	1	-2	0	x				
f	Waste Disposal	-2	-2	-2	-2	-2	x			
g	Industrial and Commercial	-2	-2	-2	0	0	2	x		
h	Agriculture	-2	-2	-2	1	0	1	-1	x	
i	Transportation	-1	-1	-2	0	0	0	2	1	x
j	Military and Strategic	-2	-2	-2	0	-1	0	1	-1	1

Table 7-5 Compatibility matrix (Kamphuis, 1997)

7.5.2 Management practice

In addition to tools like the compatibility matrix, we have the management principles, shown in Table 7-6 and the management issues shown in Table 7-7.

The coast is dynamic and policies must reflect this
Management boundaries should reflect natural processes
Conflict cannot always be resolved, so planning and legislation is required
Conflicts change with time, so a flexible management framework is required

Table 7-6 Management principles (Townsend, 1994)

Frameworks (Conceptual and Computational)	Geographic Information System
Tools	Zoning Regulations and Enforcement Public Awareness and Consultation
Responsiveness	Legal Considerations Economics Considerations Social Considerations Other Scientific and Technical Disciplines Many Jurisdictions involved

Table 7-7 Management issues (Townsend, 1994)

Geographic Information Systems (GIS) are recommended as the *conceptual/computational framework*. On the geographic base all pertinent data are stored including the locations of buildings and infrastructure, coastal protection structures, sewerage outfalls, property ownership, legal jurisdictions and physical conditions such as flood and erosion hazards, sediment sources and sinks.

The *tools* by which management is effected are: Zoning, Regulation Enforcement, Public Awareness and Consultation. These tools should be carefully selected and sharpened, showing sensitivity to the projects and the physical environments involved. They need to be incorporated into an appropriate decision-making process and a responsive management framework.

Legal, economic and social considerations and the involvement of many disciplines require responsiveness to and cooperation with other bodies, which may not think the way we do.

Every participant in matters of coastal zone management has a particular reason for being involved, and often more importantly a particular field of interest. Government agencies may be involved in the planning process for several reasons:

- Many of the resources are public property; this can lead to exhaustion of them
- Most uses of the resources will have adverse effects on other uses/users
- The pricing and requirements like a clean environment and basic human needs are a political issue
- Impartiality in the allocation of scarce goods and services must be safeguarded.

In the decision making process, the following parties may be involved:

- Government agencies
- Ministries
- Provincial authorities
- Regional water boards
- Towns
- Individuals, and corporate and private interest groups

It is necessary to have a legal and institutional framework, through which the allocation of tasks and responsibilities is made. Possible elements are: international agreements (multilateral or bilateral), national regional and local legislation and the transfer of responsibilities to a single existing or newly created agency. One of the agencies concerned may be given a leading role. Institutional changes are generally slow and do not provide a suitable way to improve coastal zone management. Most essential is the political desire to improve the management of the system.

One of the most common ways to judge proposed changes in the infrastructure of coastal zone is the execution of an environmental impact assessment (Dutch: MER) carried out according to the specifications given in the aforementioned legislation.

7.6 Building with Nature

In the context of Coastal Zone Management, conflicts of interest are often highlighted. In consequence of this, the execution of large engineering works in the coastal zone is almost automatically considered as a threat to ecology and environment. This may certainly be true in some cases, but is not necessarily true in all cases.

In the first place, it is not economical to make a design that is not in harmony with the natural conditions. It makes little sense to design a navigation channel in an area with heavy siltation or to reclaim land in an area that is subject to severe erosion. The objective of sound coastal engineering practice is to plan the works in such a way that they fit best in the natural system thus avoiding extremely high construction and/or maintenance cost.

In the second place, it is often possible to study the historical or geological developments in an area and to try to plan the works in such a way that they more or less anticipate future natural developments. In this respect, reference is made to the creation of the so-called "van Dixhoorn triangle", just North of Hook of Holland. After the extension of the breakwaters in Hook of Holland and the construction works for Europoort/Maasvlakte (around 1975), the excess quantity of sand that was available was pumped into a triangular area just north of the breakwater. The purpose was to create a new coastline that was expected to form anyway. Sufficient sand was pumped into the area to permit leaving it to nature for some time. By now, natural dunes have formed with valuable natural vegetation.

Sometimes such possibilities are not anticipated, but they occur by natural processes or by shear coincidence. An unforeseen possibility developed near IJmuiden. Due to the expected accretion south of the port, a large beach plain developed. Long after construction of the harbour entrance, the idea came up that this naturally reclaimed land could be used for recreational purposes. Now there is a large marina with hotels and other recreational facilities. However, part of the beach plain was reserved for further natural processes and it is interesting to watch the development of young dunes and pioneer dune vegetation in this area. Another valuable but unforeseen ecological development took place on one of the polders in Lake IJssel, Flevoland. Due to changes in the design of the polder, an area near Lelystad was not given a firm designation in the zoning plans. Because it had no firm designation, it was left to nature during the initial development of the polder. During this period, the area, now known as the Oostvaardersplas, grew into an important protected bird and wildlife sanctuary.

In the third place, it is possible to replace traditional shore protection techniques by new methods that are in closer harmony with nature. Such techniques should enhance rather than prevent the creation of a sound and sustainable interface between land and water. Examples of this kind of activity are the creation of natural banks (in Dutch: natuurvriendelijke oevers) and soft methods for shore protection such as beach replenishment.

All these methods are summarised by the term "building with nature" that indicates a growing use of biological and ecological knowledge by civil engineers.

8. TIDAL INLETS AND ESTUARIES

8.1 Introduction

The importance of rivers to both nature and people can hardly be overestimated. Biological activity is concentrated along and in them. Ports often originally developed along them, sometimes rather well inland. London (England), Portland (Oregon, USA), Antwerp (Belgium), Rotterdam (The Netherlands) and Hamburg (Germany) are obviously examples of such ports. In some cases, the distance to the sea has become too great an obstacle for shipping and the prosperity of the port has suffered. Deventer in the Netherlands exemplified this. Other ports have been able to move their activities downstream, much closer to the coastline (Rotterdam, Amsterdam etc.)

In this section, special attention is paid to that part of rivers, that is influenced by the tide. The tide may be perceptible over a large part of the course of a river extending a considerable distance inland. In fact, the tidal part of a river is ruled by sea levels. It is important to remember that coastal zone management measures concerning anything in the tidal estuarine area have impact throughout tidal length of a river, so the tidal part of a river must be included in the coastal zone.

8.2 Tidal inlets

A tidal inlet is a short, narrow waterway connecting a bay, lagoon or similar body of water with a large parent body of water. Tidal inlets often exist at places where there are breaks in a barrier coast.

A tidal inlet is not fixed but a dynamic entity governed by important factors such as: tidal currents, storms, the tidal prism and littoral sediment transport. Escoffier (1940) studied the stability of tidal inlets. His predominantly qualitative study led to an expression for the maximum entrance channel velocity (V_m) for a given estuary. V_m, is a function of the hydraulic radius of the channel (R), its cross sectional area (A) and the tidal range in the estuary (Δh). Since this calculation is made for a given estuary, other variables such as the channel bed roughness, its length, the surface area of the estuary, and the tidal range at sea have then all become more or less constant.

Escoffier combined the variables for a given estuary into a single parameter, x, such that a larger entrance cross-section results in a larger value of x. Qualitatively, he found that V_m varied as a function of x more or less as shown in Figure 8-1.

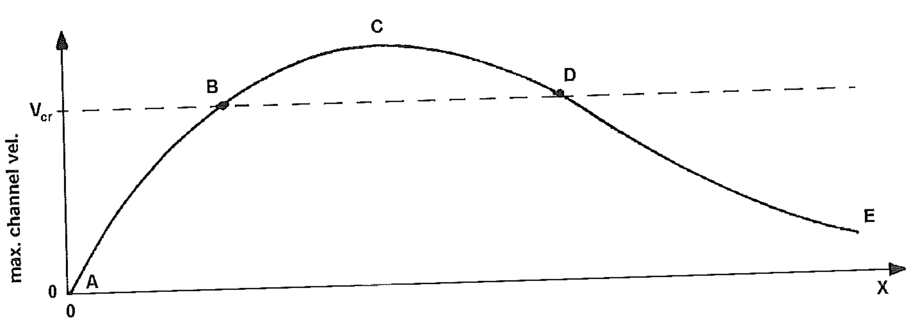


Figure 8-1 Channel velocity geometry relationship

In the range from A to C on this curve, the entrance channel is so small that it chokes off the tidal flow so that the tidal difference within the estuary will be less than at sea. On section C-E of the curve this is no longer true and the maximum current velocity decreases as the channel becomes larger.

Escoffier's next step was to introduce the concept of a critical maximum velocity V_{cr}, below which the velocity in the channel is too low to cause erosion. This critical velocity is more or less independent of the channel geometry, according to Escoffier, and he plotted it as a horizontal line on Figure 8-1.

The fate of an estuary can now be predicted by examining the curve ACE in relation to V_{cr}. Obviously, if V_m is always less than V_{cr} (for all values of x) then any sediment deposited in the entrance will remain there and the estuary will be closed off eventually. However, if a curve of V_m versus x intersects the V_{cr} line as shown at B and D in Figure 8-1, then a variety of situations can exist. If for example, the channel dimensions place it on section A-B of the curve in Figure 8-1, then the channel is too small and the friction too high to maintain itself; so it will be closed by natural processes. If the channel geometry places it on section D-E of the curve, it will also become smaller, but as it does so, the velocity, V_m, will increase; sedimentation continues until point D is reached. Lastly, if the channel configuration places it on section B-D of the curve, then erosion takes place until point D is again reached; point D represents a stable situation.

With this insight, it is now possible to evaluate the influence of changes in an estuary mouth. Since point D represents a naturally stable situation, most natural estuaries will tend to lie more or less in that region. Of course, a severe storm can cause severe sedimentation, largely filling the entrance, which is then suddenly in the state represented by section A-B of the curve. In such a situation, immediate dredging is called for to prevent complete closure. It is not necessary to restore the original situation, however, since once the entrance geometry places it on section B-C-D of the curve in the figure, nature will do the rest of the work given enough time.

Shipping interests may make it desirable to enlarge the entrance of a given estuary to accommodate larger ships. If such an expansion scheme places the channel on section D-E of the curve, the dredging industry will remain profitable for the foreseeable future. It may be possible to carry out the expansion and still prevent continual dredging by changing the channel alignment and artificially constricting its width - techniques often used in rivers – so that the larger channel cross-section remains stable. Translating such changes into a figure such as Figure 8-1 means that a new curve of V_m versus x has been generated which generally yields a slightly higher value of V_m for a given x value. This results in point D, the equilibrium situation, being moved to the right in the figure.

One of the most important questions to be answered in order to use the approach by Escoffier outlined above is "what is the stable equilibrium condition of an estuary?" or in other words, "when has point D in Figure 8-1 been reached?". Several investigators including O'Brien (1969), Jarret (1976) and Shigemura (1980) have devoted special attention to the determination of the equilibrium cross sectional area of an estuary entrance. The results for sandy coasts do not differ very much from those of O'Brien (1969). He made use of frequent surveys of inlets on the North Pacific Coast of the United States. He found that the minimum equilibrium cross sectional area of the entrance, A, was linearly related to the volume of the tidal prism. In equation form:

$$A = 6.56\times10^{-5}\, P \tag{8.1}$$

in which:

A = the minimum equilibrium cross section of the entrance channel (throat) measured below mean sea level in m^2

P = the tidal prism volume in m^3

In this equation, P, the tidal prism, is the storage volume of the estuary between low tide and high tide level. It is usually determined by multiplying the mean surface area of the estuary by the mean tide range in the estuary. Since a river flow will also contribute to the filling of the tidal prism, this volume is not equal either to the time integral of the inflow during flood or to the outflow during ebb. The coefficient is *not* dimensionless. Indeed, it has dimensions of $1/L$.

Tidal prism volumes in O'Brien's data ranged from about 8.5×10^6 m^3 to about 3.4×10^9 m^3. There is some indication that equation (8.1) would tend to yield too great a cross sectional area for smaller tidal prisms.

O'Brien also found that the bed material size had little influence on equation (8.1). Furthermore, the equation seemed equally valid for both large river mouths and for bays and tidal lagoons. However, a restriction is that equation (8.1) is valid only for inlets with a predominantly semi-diurnal tide. This piece of tidal information, combined with the realisation that the tidal prism is filled and emptied once during each tidal period of 12 h 25 m, leads to a simple but only rudimentary conclusion. This conclusion is that the average of the absolute value of the current velocity in the estuary mouth is constant, a result which certainly does not disagree with that of Escoffier more than 30 years earlier!

8.3 Tidal channels

In natural channels, the deepest channel sections develop along the outside of the river bends with the channel shifted somewhat downstream from the shoreline bend. The presence of tidal action, modified this. In a narrow channel, the influence of the tide on the river bathymetry is such as to make the location of the deeper channel in the river bend correspond more closely to that of the shoreline bend. This position represents a compromise between the development to be expected with only an ebb current and that expected with only a flood current.

In areas where the width of the river is not restricted, an entirely different pattern can develop. These tidal river reaches often have two rather independent channel systems: the ebb current concentrates in one set of channels while the flood current is often strongest in a different set of channels. Flood channels can usually be recognised because they tend to be shallower than ebb channels and they tend to die out; they lead to progressively shallower water and finally spread out on a shoal. Conversely ebb channels are continuous and tend to be deeper.

One factor that helps to account for the characteristic differences between flood and ebb channels is that the quantity of water discharged during the ebb is always greater than the quantity entering the river during the flood. This is because the river runoff, plus the ocean water that entered during the flood tide, must be discharged during the ebb. This tends to make ebb currents stronger, and hence, ebb channels deeper. An example of this is shown in Table 8-1 and in Figure 8-2, which shows the current at Rotterdam. If a similar graph were made of the current some distance further upstream, the ebb current would become more important. At some point, the current will always flow downstream at a velocity that varies according to the tide.

time (hrs)	0	1	2	3	4	5	6	7	8	9	10	11	12
aver.curr. (m/s)	-0.15	+0.08	+0.60	+0.75	+0.44	+0.07	-0.44	-0.73	-1.03	-1.05	-0.85	-0.52	-0.30
tide level (m)	-0.69	-0.50	-0.03	+0.52	+0.91	+1.04	+0.91	+0.61	+0.25	-0.15	-0.47	-0.58	-0.62

Table 8-1 Tide and current data Rotterdam

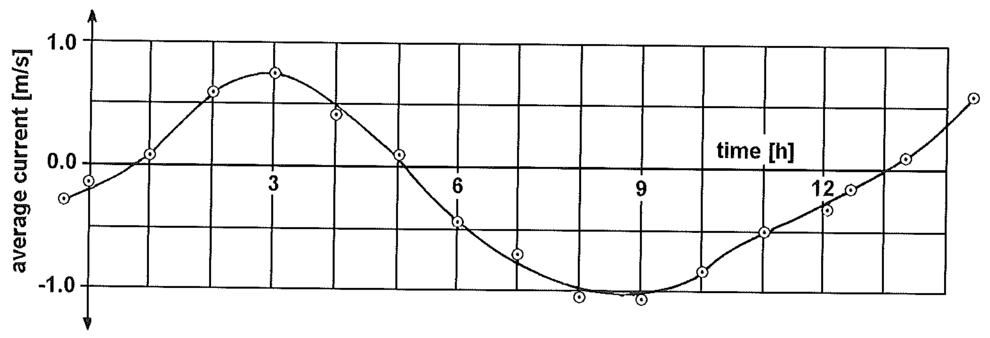


Figure 8-2 Current at Rotterdam

Another phenomenon in a tidal river is a tide-dependent variation in waterlevel. The current and the water level are related by the equations used to describe long waves. In such a case, conservation of momentum yields:

$$U\frac{\partial U}{\partial x}+\frac{\partial U}{\partial t} = -g\frac{\partial z}{\partial x}-g\frac{U|U|}{C^2 h} \tag{8.2}$$

in which:

C = Chezy coefficient
g = acceleration of gravity
h = depth
t = time
U = flow velocity
x = co-ordinate measured along the river
z = absolute water surface elevation

In this equation, it has been assumed that the river slope is small and the runoff is negligible. If the friction – the last term is equation (8.2) – is also negligible (which can be the case with a short surface wave or with a tide in the deepest ocean basin), the vertical tide (water level) and the horizontal tide (current) are in phase with one another as shown in Figure 8-3.

In a real situation, the friction term in equation (8.2) will be relatively large with respect to the inertia terms. Since in such cases some of the momentum is then lost to friction, the velocity will be reduced. Figure 8-4 shows the relationship between the vertical and the horizontal tides at Rotterdam. The current curve is the same as that in Figure 8-2. The times of high tide and low tide are indicated as well as the times of slack water (zero current).

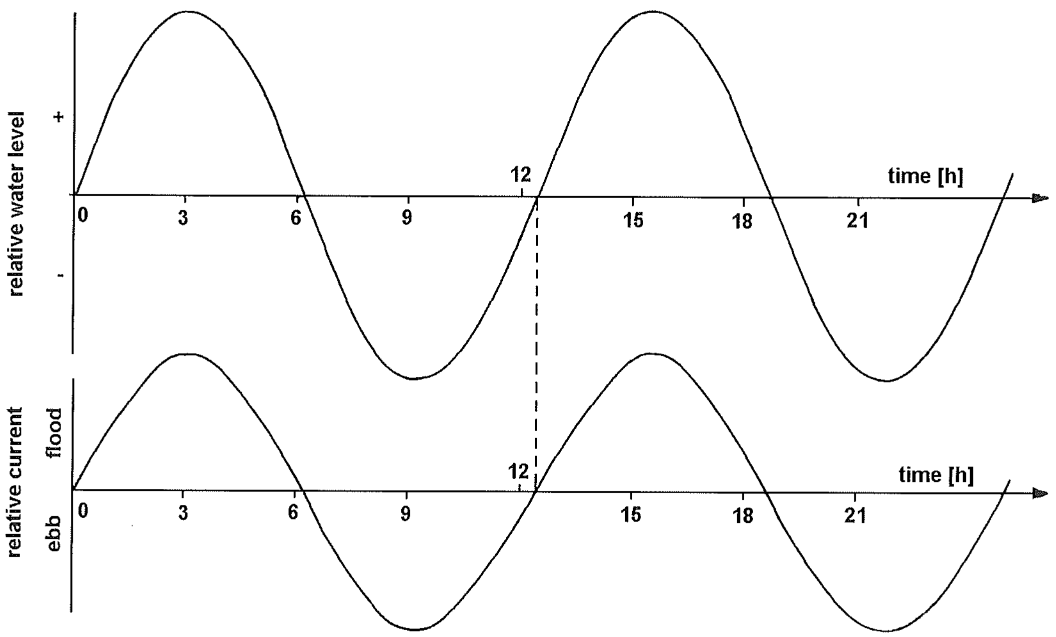


Figure 8-3 Idealised velocity-level relationship

Note that the low tide slack comes much later relative to low water than is the case at high tide. This is partially caused by freshwater river flow acting to fill the portion of the tidal prism that is furthest inland during a rising tide. This enhances the development of a water surface slope to retard the tide wave, while at low water, the river flow tends to prolong the ebb current.

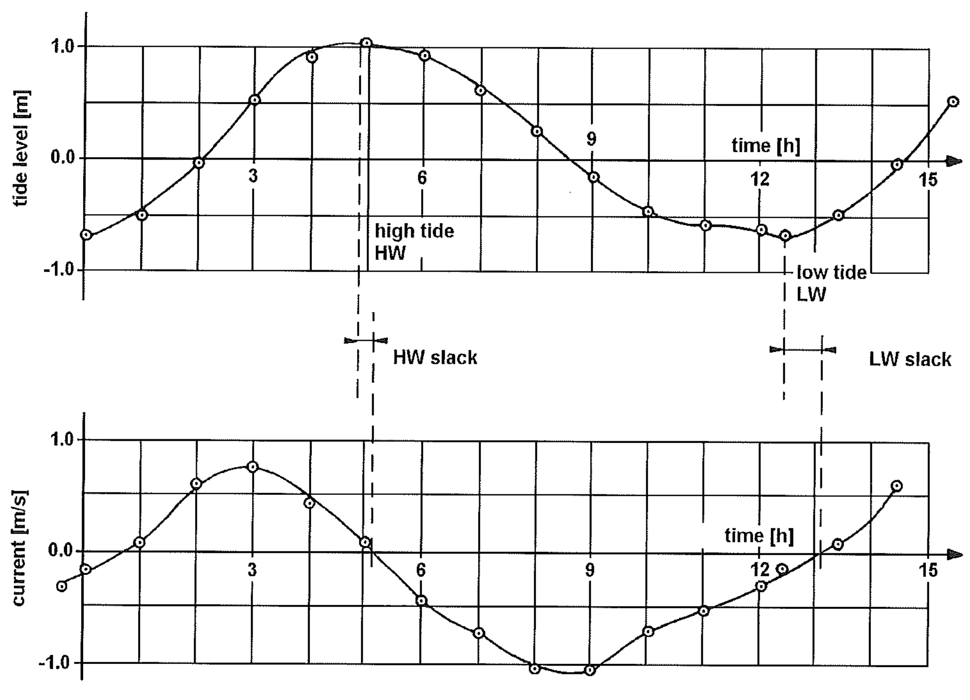


Figure 8-4 Vertical and horizontal tide in Rotterdam

A second reason why ebb channels are deeper and more continuous than flood channels is indicated in Figure 8-4. Note that the maximum ebb current occurs when the tide level is lower than that corresponding to the maximum flood current. The combined effect of higher total ebb flow and the lower stage during this flow tends to increase the velocity and enhances erosion in ebb channels. In principle, one can understand these phenomena by adding up a hypothetical sinusoidal tidal current and a constant river discharge (Figure 8-5). Conversely, it is possible to derive the tidal prism and river discharge from the measured discharge curve.

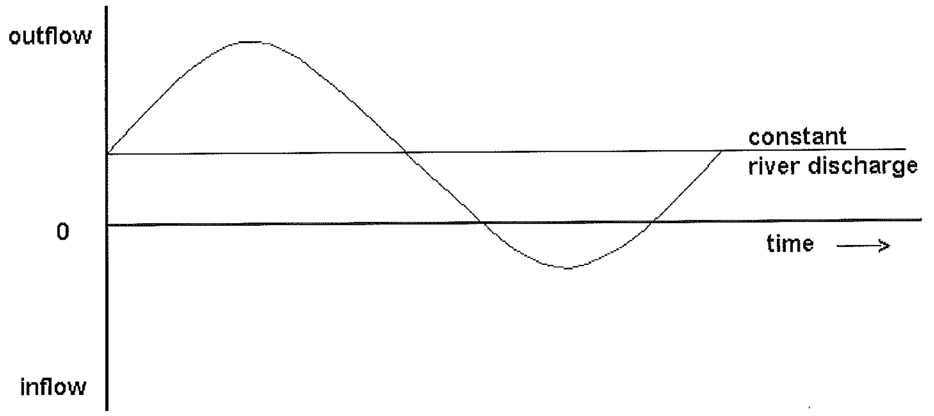


Figure 8-5 Combined effect of tidal flow and river discharge

9. POLLUTION AND DENSITY PROBLEMS

9.1 Introduction

In the coastal zone, many engineering problems are related to the differences between saline and fresh water. Another common cause of problems is pollution. This chapter deals with both.

9.2 Pollution

9.2.1 Types of pollution

Pollutants include:
1 Human wastes
2 Oil
3 Halogenated hydrocarbons
4 Other organic materials
5 Heavy metals
6 Heat
7 Radioactive materials
8 Fine sediment

Human faecal waste is often considered first, since it raises such a great aesthetic problem. Nevertheless, it is certainly a natural product and faecal waste is produced in great quantities by marine life. According to Bascom (1974-1), six million tons of anchovies off the California coast produce as much faecal material as 90 million people though not necessarily containing bacteria dangerous to human life. Seawater with a high faecal content provides food for lower forms of marine life, which in turn provide food for what is generally considered desirable marine fauna. However, two aspects of the disposal of faecal wastes remain important: oxygen consumption from the water, and bacteria. The oxygen demand can reduce the dissolved oxygen level below the level that is needed by marine life. While most bacteria are soon killed by contact with seawater (within hours), this is not necessarily true of all types; thus epidemiological problems may arise.

Oil and petroleum products are perhaps the most controversial pollutants. The public reaction to oil spills by ships is usually emotional and vehement. Shipping is not the only source of marine oil pollution, however. Unknown quantities of oil seep into the oceans naturally. A report compiled for the Connecticut State Legislature concludes that more than two thirds of the oil discharged by man into the seas comes from automobile engines and oil sumps of other machines. This oil causes no great problems, since the rate of input is low enough and it is sufficiently dispersed to be broken down by natural processes, which is not the case if an oil tanker is damaged. Oil pollution from major spills often is a temporary problem. The short-term biological influences can be severe, but the pre-existing natural situation usually restores itself within a few years. This is not true for the next category of pollutants.

Halogenated carbons include the most common organic pesticides. While a few of these chemicals, such as TEPP lose their lethal properties rather quickly, others such as DDT seem to be virtually indestructible in nature. The process of the concentration of pesticides in certain types of marine life (bio-accumulation) is rather well known and quite alarming.

The discharge of nutrients into bodies of water may have stimulating effects on the marine life.

However if uncontrolled, this soon becomes a way of over stimulating, which can be disastrous to the ecological equilibrium. Oxygen is consumed in the biodegradation of the nutrient materials. Obviously the last word about this item has not yet been spoken.

Because of the electrostatic properties of clay, fine sediments may bind heavy metals and extremely long molecules (halogenated hydrocarbons) that are present in the water column due to natural causes or human activity. Therefore, the concentrations of heavy metals (and other organic components) in fine sediments are relatively high. As long as the pollutants are bonded to the sediment, they cause relatively little harm. The binding force may be lost, however, due to strong mechanical action (turbulence) and changes in the chemical and physical conditions (acidity, salinity, presence of oxygen, temperature). In such cases the pollutants become available in high concentrations, and they can easily be introduced into the biological cycle. Uncontrolled discharge of heavy metals has led to serious environmental disasters, among others in Japan, where Itai-Itai and Minimata disease have affected the human population.

It is extremely difficult to separate the heavy metals from the large quantities of silt in the estuaries. Therefore, emphasis is placed on the reduction of the emissions on one hand, and controlled storage of polluted sediments on the other hand. Examples: The Slufter basin in the port of Rotterdam, Ketelmeer. Heavy metals also enter the sea from the atmosphere. Forest fires, for example, add metallic oxides to the atmosphere, which deposits them over the whole world. Just as with discharges of many pesticides, the influence of heavy metal discharges is cumulative. An example of the cumulative action as influenced by man is shown in Figure 9-1, which shows the lead concentration in layers of sediment in the ocean near Long Beach, California. The sharp rise in concentrations in recent years is attributed to airborne lead from automotive emissions.

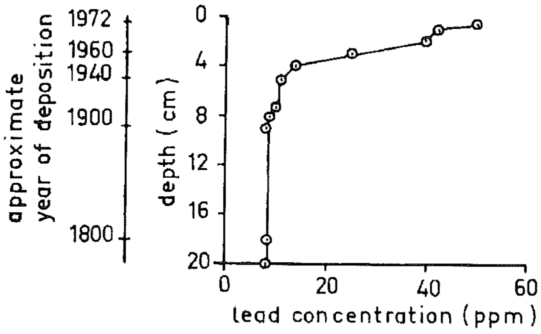


Figure 9-1 Lead concentration in sediment, Bascom (1974-1)

Thermal emissions may be either warmer (power station cooling water) or cooler (liquefied natural gas conversion) than the surrounding water. Most marine life can adapt to the modified thermal climate near such a heat source or sink, but are often killed either mechanically or as a result of abrupt temperature and pressure changes as they are drawn through the plant. Heat discharged into the oceans is only of local biological significance.

Radioactive wastes form the seventh category of pollutants. Marine life can tolerate a larger radiation dose (before it becomes fatal) than man. Man can conceivably ingest a fatal dose of radioactive poisons from seemingly healthy fish. Therefore, radioactive wastes should not be put into the environment of fish (the sea). It is not a good solution to dispose of wastes into subduction sinks, because the natural recycling processes in the deep water are very slow.

Fine sediment itself, as residue from dredging, can be a danger for marine life in certain locations. High concentrations of suspended clay particles can inhibit the penetration of sunlight into the water which may be disastrous to certain species although in other places this may not be a problem. So sediment has two ways of forming a threat to the environment: by reducing light penetration and by carrying other pollutants such as heavy metals.

9.2.2 Control measures

Legal sanctions that are attainable and consistent provide good control measures. Environmental assessment of plans can be a control instrument. Common problems during environmental assessment are:

1 effects on man, flora, and fauna are often indirectly related to direct consequences of an activity
2 direct consequences of an activity are often not easy to quantify
3 direct consequences are measured in different units
4 direct consequences are not easy to express in terms of money
5 alternatives are evaluated on their final effects, and temporary effects are often omitted/not considered

These problems could possibly be met by creating a social basis for the evaluations. Effects should be quantified and expressed on the same basis and in the same units. Temporary effects should not be forgotten. In a project, specific attention must be given to the issue of responsibility. Companies that produce pollutants, often fail to register waste disposal properly. Frequently this activity is contracted out to a cleaning company, which may create legally unclear situations.

Many pollution problems cross the borders of a country. In the case of river pollution, polluting companies which are situated upstream cause problems in downstream river sections so coastal zone management practice should include the upper part of the river. In the case of air pollution is even more complicated and many environmental problems must be tackled and international measures must be applied. In particular the different standards used by neighbouring countries generate unclear situations and political dissatisfaction.

During the last forty years, international legislation has been developed with respect to the marine environment. This started with the Treaty for the Continental Shelf (1958), which determines the rights of coastal countries, and also obliges them to take protective measures for marine life. The London (Dumping) Convention (1975), which has many signatories, contains a periodically updated black list (chemicals which may not be dumped or burnt at sea) and a grey list (dumping/burning only with a permit). In order to prevent ships from polluting activities, the MARPOL-agreement (1982) has been widely accepted. The Convention of the United Nations on the Right of the Sea (1982) includes the legal framework for the worldwide use of seas and oceans. Rules concerning the conservation and management of marine life, and protection and conservation of the marine environment are included in this agreement.

9.3 Density currents in rivers

Up to this point, tidal influences on rivers have been considered without regard to the fact that the river water is relatively fresh while the ocean water is relatively salty. Salinity variations cause variations in water density, just as do temperature variations (as discussed in Chapter 4).

Density differences cause additional currents, while variation in salinity can affect the physical chemistry of fine sediments.

9.3.1 Salinity variations with tide

Let us consider the salinity profiles found in rivers. Salinity profiles can be drawn by using haloclines (lines of equal salinity). If the haloclines are vertical, one speaks of a well-mixed condition. Horizontal haloclines indicate a stratified condition. In Figure 9-2, longitudinal sections of an estuary show different salinity distributions (‰).

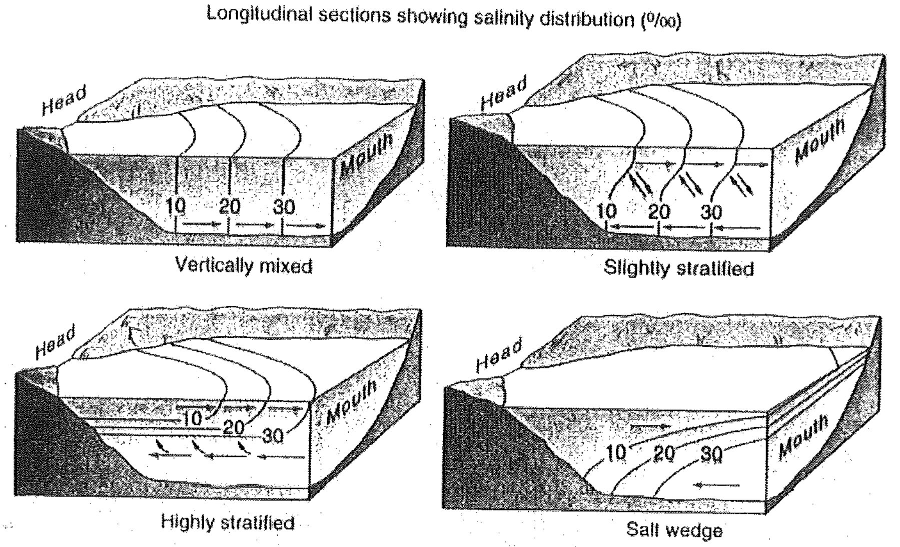


Figure 9-2 Different salinity distributions

The basic flow pattern in an estuary is a surface flow of less dense fresh water toward the ocean, and an opposite flow of salty seawater into the estuary along the bottom. The dimensions of each flow, and the degree of mixing between the two, depend on specific conditions in each estuary. Note that, in the northern hemisphere, surface fresh water flowing towards the mouth of the estuary extends much further seaward along the right-hand shore as one faces seaward. This is due to the Coriolis effect. The opposite side of the estuary experiences a greater marine influence. In most estuaries, the marine water inflow occurs in the subsurface.

Unless there is more than enough fresh water flow in the river to completely fill the entire tidal prism during the rising tide phase, salt water enters an estuary during a rising tide. Few rivers have sufficient flow over the entire year to prevent the intrusion of salt water at least occasionally. Indeed, the opposite is more often true; there is seldom sufficient flow to prevent the intrusion of salt water.

The salinity at any point in a river can be expected to vary according to the tide. Since the salt water comes from the sea, the maximum salinity should also be expected at about the time of the high water slack. This is illustrated for Rotterdam in Figure 9-3. The current data is the same as that in Figure 8-2. Again, flood currents are considered positive.

Recalling from Section 4.2 that seawater has a salinity of about 35 ‰ we see that pure seawater never really reaches Rotterdam. Mixing has already dispersed the incoming seawater through the fresh river water forming a brackish mixture. If we were to measure salinity at a point nearer to the sea, then we could expect to find higher maximum salinity values.

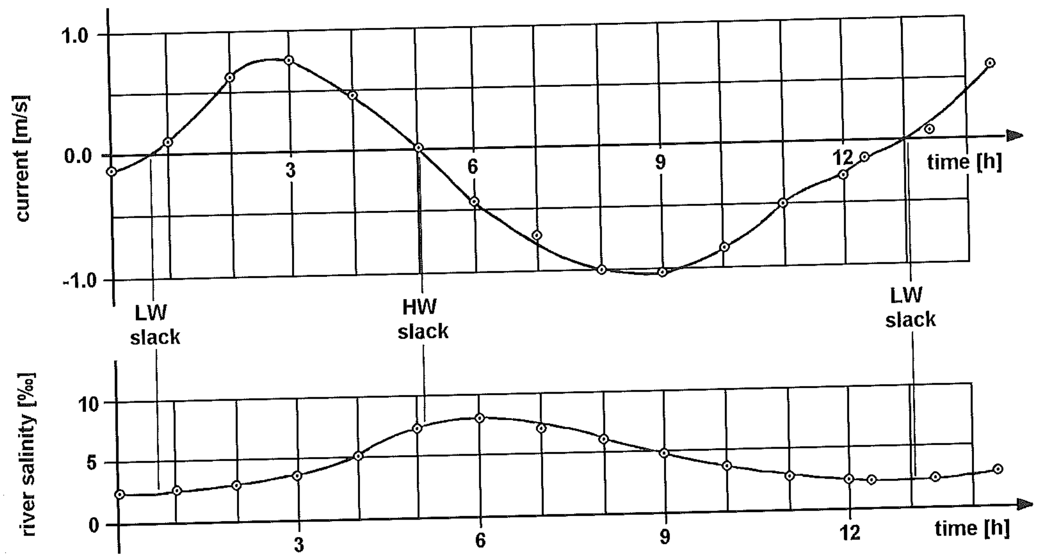


Figure 9-3 Current and salinity at Rotterdam

The degree of mixing in an estuary can be approximately related to the ratio between the volume of the tidal prism and the river flow, named the mixing parameter (M).

$$M = \frac{Q_r T}{P} \tag{9.1}$$

in which:
M = the mixing parameter [-]
P = the volume of the tidal prism [m^3]
Q_r = the fresh water river flow [m^3/s]
T = the tide period [s]

For a well-mixed estuary M goes to 0 while a well-stratified estuary M becomes 1.

Comparison of the River Schelde and the Rotterdam Waterway, indicates that the conditions at Rotterdam show stratification (higher river discharge, lower tidal range) and the Schelde has a more mixed character.

A more fundamental approach to the problem used by Ippen and Harleman (1961) investigates the mixing process through use of a dimensionless stratification number (S). This is defined for a unit mass of fluid as:

$$S = \frac{\text{rate of energy dissipation}}{\text{rate of potential energy gain}} \tag{9.2}$$

The energy dissipation in the numerator results from the damping of the tidal wave in the estuary; the denominator reflects the potential energy gain as water increases in density (salinity) moving downstream.

Harleman and Abraham (1966) related the stratification number uniquely to a dimensionless estuary number (E), defined by:

$$E = \frac{pF^2}{Q_r T} = \frac{F^2}{M} \tag{9.3}$$

in which:

F = the Froude number based upon the maximum flood current velocity at the estuary mouth $\left(u/\sqrt{gh}\right)$

M = mixing parameter

The estuary number (E) has the advantage over the stratification number (S) that its parameter can be rather easily evaluated. In contrast to the mixing parameter (M), estuary mixing increases with increasing estuary number values. Well-mixed estuaries have estuary numbers greater than about 0.15.

9.3.2 Static salt wedge

In a fresh water river discharging into a saline sea, a salt wedges forms (see Figure 9-4). The sea water intrudes along the river bottom under the fresh discharge water. The length of the intruding wedge is determined by the equilibrium between the friction, τ_I, along the interface and the horizontal pressure gradient resulting from inclination of the interface. When this equilibrium is strictly satisfied, the salt wedge will be in a stable position with the fresh water flowing seaward on the surface and spreading out in a thin surface layer at sea. Schijf and Schönfeld (1953) derived an expression for the length of such a wedge in a prismatic, horizontal, rectangular channel discharging into an infinite, non-tidal sea.

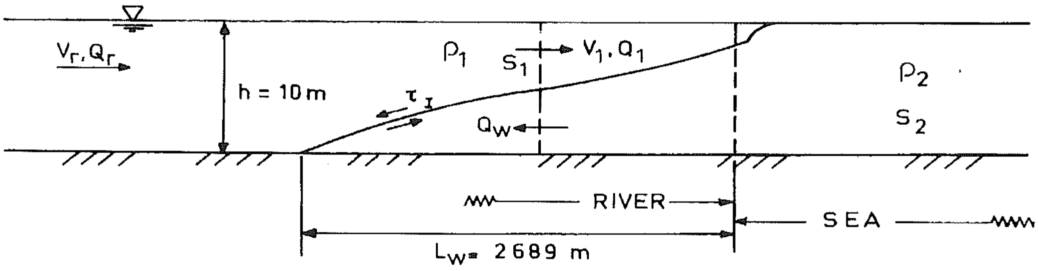


Figure 9-4 Static salt wedge in river mouth

If no mixing occurs across the interface, then their equation is:

$$L_w = 2\frac{h}{f_1}\left[\frac{1}{5F^2} - 2 + 3F^{\frac{2}{3}} - \frac{6}{5}F^{\frac{4}{3}}\right] \tag{9.4}$$

where:

$$f_1 = \frac{8\tau_I}{\rho(V_1 - V_2)V_1 - V_2} \tag{9.5}$$

and:

$$F = \frac{V_r}{\sqrt{\delta g h}} \tag{9.6}$$

where:

L_w = length of wedge [m]

V_r = velocity in the river upstream of the wedge [m/s]

V_1 = velocity in the fresh water above the wedge [m/s]
V_2 = velocity in the salt wedge [m/s]
τ_l = friction stress along the interface [N/m^2]
δ = relative density of the water masses $((\rho_2-\rho_1)/\rho_1)$ [-]

This expression illustrates the influence of water depth (h), the river discharge velocity (V_r) and the density difference on the salt intrusion. A reasonable value for f_i is in the order of 0.1. Of course, in the idealised equilibrium state, $V_2 = 0$. This is why no friction stress on the bottom is shown in Fig.6.4. The data used to plot this figure were: $f_i = 0.08$; h = 10 m; V_r = 0.2 m/s; and δ = 0.0246, giving L_w = 2689 m. The figure is drawn with a distortion of 1:100.

In a real situation there is a state of dynamic equilibrium. Mixing will take place along the interface between the water masses. Salt and sea water will be transported with the river water back to the sea. This is indicated in Figure 9-4 at the vertical dashed line half way along the wedge. Since the total net flow out of the river must be equal to the fresh water runoff:

$$Q_1 = Q_r + Q_w \tag{9.7}$$

where:
Q_w = inflow in the wedge
Q_r = fresh water river flow
Q_1 = net outflow through the cross section

Continuity of salt content must also be maintained. This implies that:

$$Q_1 S_1 = Q_w S_2 \tag{9.8}$$

where S_1 and S_2 are the respective salinities.

When different values of V_r are substituted into equation (9.4) (via (9.6)) it is found that L_w decreases as V_r increases; indeed, F = 1 yields L_w = 0. Remembering that increasing V_r also implies an increasing Q_r we seem to discover a contradiction to the rules of thumb presented in equation (9.1) and (9.3). According to that, increasing Q_r should lead to a more stratified estuary, and hence, a longer instead of a shorter salt tongue (wedge). This dilemma is explained by realising that all tidal influences have been neglected in formulating equation (9.4); thus, this comparison is invalid.

In a real estuary, the salt wedge intrusion problem is much more complex. The river flow, Q_r, varies, tidal influences are present, and the estuary is certainly not prismatic.

Usually, the tidal influence is most important - it leads to an oscillatory motion of the entire two-layer system over an uneven bottom. This motion, of course, increases mixing across the interface. Indeed, in estuaries with a strong tidal influence and little fresh water flow, stratification can be essentially destroyed, leading to a well-mixed estuary. At any given time and place there is little vertical salinity gradient.

9.3.3 Horizontal stratification

A horizontal stratification implies a horizontal interface between two layers. If the upper layer is less dense than the lower layer this stratification will be in equilibrium. In fact, such an interface can remain stable even though both layers of water are in motion. This stratification, caused either by salinity or temperature differences is found in the oceans and in all but the shallowest lakes.

When a horizontal stratification surface exists within a body of water, waves can be generated at this interface, just as on the upper surface. Indeed, the upper surface of a body of water is also an interface between two media (water and air). However, for internal waves on an interface between water layers, the density of the upper fluid is nearly the same as the density of the lower fluid. The resulting low density difference will have a strong influence on the phenomena involved, especially when these are compared to wind waves.

"Dead water" is a phenomenon that is related to horizontal stratification; the situation in which a generally stable salt layer is lying under a fresh layer. Internal waves (Figure 9-5) can be caused by a disturbance such as a ship, earthquake or underwater landslide. They can also result from shear forces along an interface between two layers in relative motion.

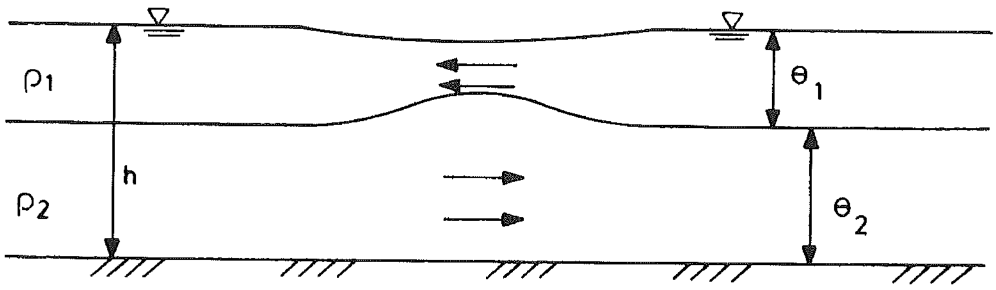


Figure 9-5 Internal wave.

The celerity of a wave on an interface is given by:

$$c=\sqrt{\frac{(\rho_2-\rho_1)g\theta_1\theta_2}{\rho_2\theta_1+\rho_1\theta_2}} \tag{9.9}$$

where:
c = wave speed
ρ = density
θ = layer thickness

As ρ_2 is nearly equal to ρ_1 in Equation (9.9), it can be approximated by:

$$c\approx\sqrt{\frac{(\rho_2-\rho_1)g\theta_1\theta_2}{\rho_1 h}}\approx\sqrt{\frac{\delta g\theta_1\theta_2}{h}} \tag{9.10}$$

where:
δ = relative density = $(\rho_2 - \rho_1) / \rho_1$
h = total depth = $\theta_1 + \theta_2$

These waves can be very high, since the gravitational influence on them is small. They are accompanied by much smaller negative waves on the water surface. Indeed, as a first approximation, the ratio of surface wave height to internal wave height is equal to δ. These internal waves can absorb a considerable energy from a ship causing the so-called "dead water".

This is explained via an example. A ship of 4 m draft sails into a stratified harbour with a 3 m thick surface layer of relatively fresh water (salinity S = 5 ‰ and temperature T = 2 °C) above a deeper layer of 7 m thick with S = 36‰ and T = 4°C . What is the maximum speed that this ship can attain?

$$\begin{aligned}
&\sigma_{t1}=4.00: \quad \rho_1=1004.00\,kg/m^3;\\
&\sigma_{t2}=28.70: \rho_2=1028.70\,kg/m^3;\\
&\theta_1=3\,m, \quad \theta_2=7\,m;\\
&c=\sqrt{\frac{(1028.7-1004.0)(9.81)(3)(7)}{(1004.0)(7)+(1028.7)(3)}}=0.709\,m/s
\end{aligned} \tag{9.11}$$

The only way the ship can move faster than this wave is to cut through it or climb over it; neither is very likely! This dead water phenomenon also played a role in a naval battle some centuries ago in the area where the rather fresh Baltic Sea water flows over more dense water from the Skagerak.

9.3.4 Siltation in rivers

As has already been indicated, tide cycles cause the salt tongue or the haloclines to move back and forth in the river as a function of the tide. The most direct consequence of a salt tongue in a river is its effect on the siltation pattern of the estuary. The current along the bottom of the estuary is drastically changed by the presence of the salt tongue. Upstream of the tip of the tongue, the current along the bottom is directed towards the sea, while within the wedge there is often a small flow into the estuary. Since the bottom velocity at the tip of the tongue must be zero, it can be expected that material will be deposited there. In estuaries where there is little tidal influence and the position of the salt wedge remains relatively stable, this local sedimentation can form a pronounced shoal in the river. While so far the cause of this tongue has been attributed to salt, this phenomenon can also be found in an estuary which has a density difference caused by other factors such as thermal gradients. For example, this phenomenon might also be observed in the cooling water discharge channel from a power station, even one located on a fresh water lake.

When the suspended sediment in a river consists of clay and the density tongue is caused by salinity differences, electro-chemical processes can also strongly influence the siltation pattern in the estuary. Suspended clay in fresh water consists of flat or needle-shaped particles that have a maximum dimension of less than a few micrometers. Because of their shape, large surface area and the crystal structure of the clay minerals, these particles are negatively charged on the surface. Since the particles are so small that the electrostatic forces, rather than the gravitational forces control the behaviour of the clay particles, and work to keep the particles separated and in suspension.

As the salinity of the water increases, the positive ions (Na^+, Mg^{2+}, Ca^{2+} etc.) present tend to neutralise the electrostatic forces, thus allowing the clay particles to flocculate, and settle. A salinity of about 3 ‰ is critical in this process. The electro-chemical influences are only important when there are salinity variations below this value. The flocculation caused by an increase in water salinity is at least partially reversible. When, later in the tide cycle, the salinity decreases, the flocks of clay particles exposed to the fresh water can explode, re-dispersing the individual particles in suspension. This process can provide disturbing influences on the suspended sediment content in areas where low, variable salt concentrations can be found. An impression of the magnitude of this influence on siltation can be gained by comparing the fall velocity of clay particles in fresh water to the fall velocity of flocks of particles in salt water (S > 5 ‰). Allersma, Hoekstra and Bijker (1967) report that the apparent ratio between these fall velocities was more than 1:50.

The quality of the material forming the river bed in such an area is not the same as that of the usual compact clay. Indeed, the sediment which results from flocculation contains a large quantity of water. The volume of the sediment (solid particles plus water) can be 5 to 10 times the volume of the particles. (In soil mechanics terminology, the void ratio can be as high as 90%). Obviously, such a high volume of water will keep the mud density low (usually between 1100 and 1250 kg/m^3). The material behaves as a viscous fluid with a viscosity in the order of 100 to 5000 times that of water; this is comparable to Dutch yoghurt (except for colour). This material, called sling mud, is difficult to detect when making soundings. It appears as a faint reflection on an echogram. The sediment is so soft that ships can often sail through it. The consolidation process of such soft silt is very slow. Layers up to 2.5 m thick remain fluid for several weeks – even in a laboratory settling tube. This sling mud can be brought into suspension again when the current velocity above it reaches a critical value ranging between 0.2 and 1.0 m/s.

The upper portion of the mud layer behaves as a viscous fluid and while this is easy to pump with a dredge, its extremely low density results in poor dredge productivity measured in terms of quantity of solids moved per hour. One means of improving this situation is to dredge a deep pit so that the silt layers can move to that pit and consolidate slowly there. Mud of higher density can then be withdrawn from the deepest part of the pit using a dredge. Now, there remains only a problem of getting the mud layer to move to the pit. There are two options/possibilities:

1. If sufficient surface slope is available, gravitational force will cause the sling mud to flow towards and into the pit.
2. The second approach relies on the shear stress exerted by water flowing above the bed, (the tide, for example) to provide a driving force for the mud movement. A danger is that if the surface shear stress becomes too high, the boundary between mud and water becomes turbulent stirring the mud into suspension. Naturally mud in suspension will simply pass over the nicely prepared pit.

9.3.5 Methods to combat density currents in rivers

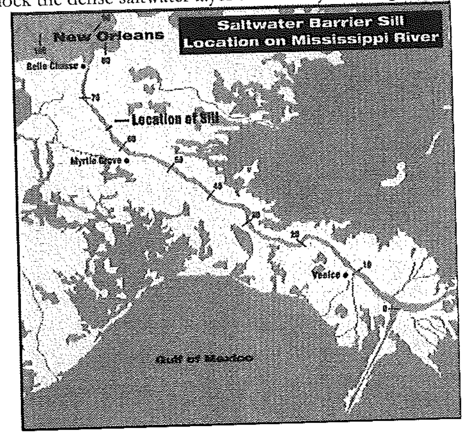
There are relatively few economical techniques that can be used to combat the intrusion of a salt tongue into a river. Many more techniques are available for more restricted areas such as harbour basins and channels. It has been indicated that the length of the salt wedge can be reduced by decreasing the water depth and by increasing the fresh water flow. In the Netherlands, the discharge of fresh water through the New Waterway has been increased as a result of the completion of the Northern part of the Delta Project (Volkerak dam and locks, Haringvliet sluice). In addition, the development of the Europoort harbour area has eliminated the necessity for bringing large, deep ships into the New Waterway past the Europoort entrance. Thus in the 1970s and 1980s, sills of gravel could be built on the bed of the New Waterway in Rotterdam. These decreased the effective depth and to a large extent drove the saltwater tongue back towards the sea. In this way, salt intrusion into the hinterland via the mouth of the "Hollandsche IJssel" near Krimpen, could be eliminated. A similar measure was taken more recently in the Mississippi River to protect the water supply intakes of New Orleans. (See insert from: "Civil Engineering", December 1999).

Thermal density currents can be combated by either enhancing the mixing of the two water layers or stimulating the heat transfer process between layers or to the atmosphere. For example, although this is not very commonly done, mixing can be enhanced, by increasing the turbulence in the thermal discharge or artificially generating an unstable stratification. Increasing the discharge velocity and constructing of a pile-supported jetty in front of the discharge flume of a power station have been suggested as means to increase mixing by increasing turbulence. Air-bubble screens, created by pumping air into the water, have a similar effect. Naturally-unstable stratification is often artificially generated when warm sewage of low salinity is discharged near the bottom of the sea. As the lighter sewage rises through the seawater, the resulting turbulence helps to disperse it. Obviously, another solution to thermal pollution problems is to re-cool the discharge water before it is released. This may be accomplished by retention in shallow pools or by circulation through a cooling tower. Sometimes, this cooling can be accomplished by simply using a long wide discharge channel. The objective in all of these solutions is to transfer the heat to the atmosphere.

During the design of cooling circuits, proper attention must be paid to the prevention of shortcuts between intakes and outlets.

Underwater Sill Impedes Mississippi River Saitwater Wedge (from: Civil Engineering. December, 1999)

The U.S. Army Corps of Engineers has constructed a 30 ft (9 m) high sill, or underwater dam, at the bottom of the Mississippi River to stop a saltwater wedge from reaching municipal water intake structures south of New Orleans. The sill extends 1,700 ft (518 m) across the river and is designed to block the dense saltwater layer that each year creeps upstream along the river bottom.



The sill is not difficult to build, says Burnell Thibodeaux, a supervisory hydraulics engineer with the Corps of Engineers in New Orleans. It is more difficult to forecast what the river is going to do. "We know we can start building [the sill] and the river could turn around," Thibodeaux says.

The Corps monitors the saltwater wedge annually, and when the wedge approaches the water supply intakes the Corps must decide whether to spend the money to build a sill or risk contaminating the water supply for more than a million people in the New Orleans metropolitan area.

The Corps awarded a $1.4-million dredging contract in September to Mike Hooks, Inc., of St. Charles, Louisiana. This is the second time the Corps has built the sill to block the toe of the wedge -the first was constructed in 1988. Both sills were built in the same location, near mile marker 64, which is approximately 30 mi (48 km) downstream from New Orleans. When construction began this year in late September, the toe of the wedge was near mile 63, The project took about 30 days to complete.

When the river flow drops below 250,000 cu ft/s (7,080 m^3/s) saltwater from the Gulf of Mexico, 104 mi (167 km) downstream from New Orleans, begins to creep upstream. During dry summers the wedge can extend farther than normal. The first drinking water intake on the river is at Belle Chasse, about 20 mi (32 km) south of New Orleans.

"The sill's location was chosen because we can use heavier than usual sands just above it that will remain in place better than finer materials," says Fred Schilling, the Corps's Mississippi River operations manager. The bottom profile also works well, he says, because the river is 80 ft (24 m) deep at that point -enough to block the toe of the saltwater wedge yet not so deep that an exorbitant amount of material would need to be dredged.

About 800,000 cu yd (611,840 m^3) of sediment was placed along the underwater "levee," which in some places is as wide as 1,500 ft (457 m). The 45 ft (13.7 m) deep navigation channel will not be affected by the sill because the top of the latter is about 50 ft (15 m) below the water surface. When the river's flow rate reaches 400,000 cu ft/s (11,328 m^3/s), the sill erodes in a matter of weeks, Schilling says.

9.4 Density currents in harbours

The tide causes ebb and flood currents in a harbour. When the traditional equations of motion are, inertial terms are less important here. This means that if no density effects are involved the current in the harbour mouth will be slack just at the times of high and low water. However, the tide also causes density currents. If such effects are involved, the density stratification at the mouth of a harbour basin just after the river salinity has changed can be outlined by a vertical interface. This configuration may be called vertical stratification of the salinity profile.

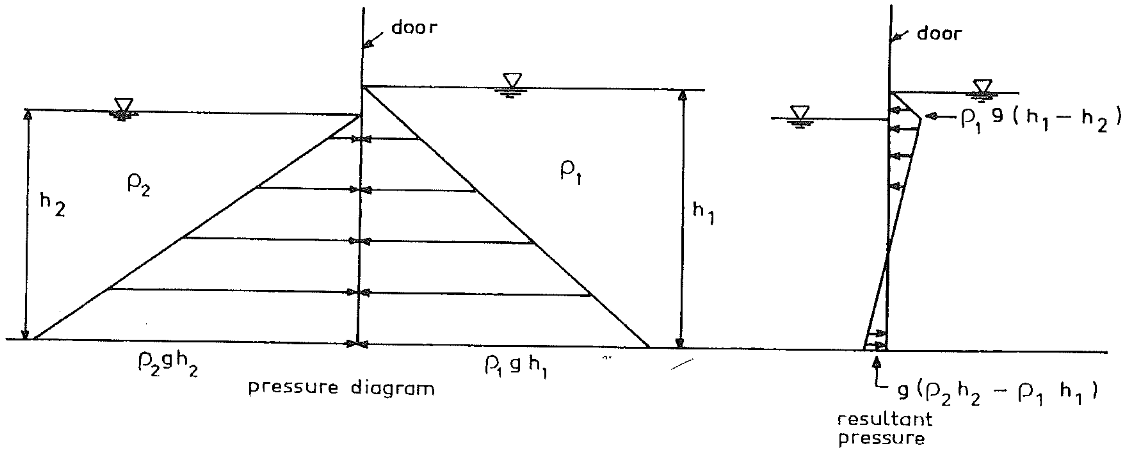


Figure 9-6 Hydrostatic pressures on each side of a lock gate separating salt from fresh water.

This situation is very much the same as that of a lock, where there is fresh water on one side and salt water on the other side. Hydrostatic pressure differs on each sides and the result is shown in Figure 9-6. Opening of the lock gate can take place when there are equal water levels on both sides of the gate. In this case, there is still a resultant horizontal force working on the gate that prevents smooth opening. The resultant force becomes zero, if:

$$\frac{1}{2}\rho_1 g\, h_1^2 = \frac{1}{2}\rho_2 g\, h_2^2 \tag{9.12}$$

where:
ρ = mass density of water
g = gravity acceleration
h = depth

When $\rho_2 > \rho_1$, then Equation (9.12) yields:

$$\frac{h_1}{h_2} = \sqrt{\frac{\rho_2}{\rho_1}} \tag{9.13}$$

While the resultant force on the gate is zero, the resultant moment on the gate is not zero! After opening the gate this condition is unstable. It therefore leads to a current pattern as shown in Figure 9-7. The flow of the denser layer can be compared to the flow of water down a river valley just after a dam has burst. This is called a dry bed curve. The toe of the dry bed curve is held slightly back by the friction along the bottom.

Since the volume of water in the lock chamber or harbour remains constant – neglecting filling or emptying – the inflow must equal the outflow caused by the density difference. Since the usual assumption is that the flow in each direction occurs over half the depth, then the two flow velocities must be equal for a rectangular channel.

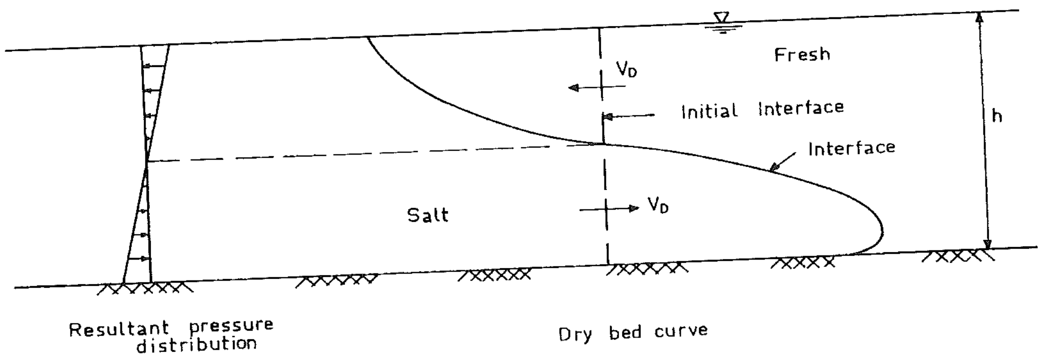


Figure 9-7 Dry bed curve

The velocity of the dense layer is:

$$V_D = 0.45\sqrt{\delta g h} \tag{9.14}$$

where:
V_D = velocity in the dry bed curve
δ = relative density = $(\rho_D - \rho) / \rho$
h = water depth

The value of the coefficient in equation (9.14) (0.45) depends on friction, and was introduced by Simpson (1987). In practice, the value of the coefficient 0,45 is a bit too high; a value somewhere between 0.3 and 0.4 usually gives better results.

In a real harbour on a tidal river (and of course also in a lock chamber) the flow into the harbour is the superposition of the filling flow and that caused by the density current. Therefore, velocity distributions can be superimposed, while the sediment transports cannot be simply added, except when the sediment concentration is constant over the entire depth (which not usually is the case).

As an example, the actual conditions in the Port of Rotterdam in the vicinity of the 2nd Petroleum Harbour are given. Previously, we have already seen how inertial effects maintain a flood current in a river even after high water. For a harbour, however, the inertia terms are much less important and the current in the harbour mouth will be slack just at the time of high and low water. This is true when no density effects are involved. Table 9-1 lists the data used to plot Figure 9-8 showing the tidal conditions in the Rotterdam Waterway in front of the 2nd Petroleum Harbour as previously mentioned. (For the time being the influences of the density current have been eliminated from the data listed in the table). Since the currents in the harbour entrance are so small. They are listed in centimetres per second. These figures are essentially the same as those listed in Table 9-1.

Time	Harbour Tide Level	River Current	Harbour Filling Current
(hrs.)	(m NAP)	(m/s)	(cm/s)
0	-0.69	-0.15	0.9
1	-0.50	+0.08	2.2
2	-0.03	+0.60	3.2
3	+0.52	+0.75	2.2
4	+0.91	+0.44	1.1
5	+1.04	+0.07	0
6	+0.91	-0.44	-1.5
7	+0.61	-0.73	-2.1
8	+0.25	-1.03	-1.6
9	-0.15	-1.05	-1.1
10	-0.47	-0.85	-1.5
11	-0.58	-0.52	-0.8
12	-0.62	-0.30	0

Table 9-1 Tidal conditions measured in the Rotterdam waterway (2e Petroleumhaven)

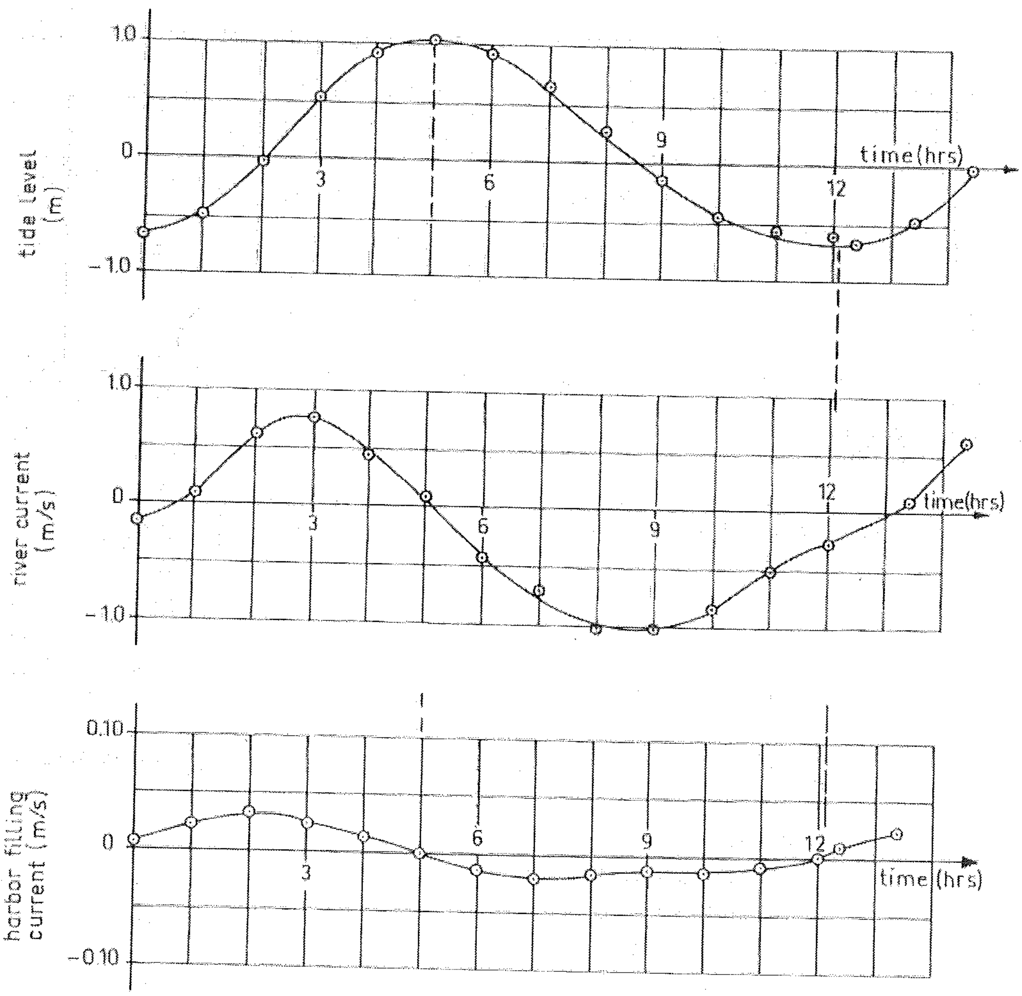


Figure 9-8 Tidal conditions as measured in the Rotterdam waterway (2e Petroleumhaven)

The density conditions and the resulting density currents measured in the mouth of the 2nd

Petroleum harbour are given in Table 9-2 and Figure 9-9. Comparing these measured data with the figures that can be calculated using equation (9.14) leads to the conclusion that there are some important differences between theory and practice.

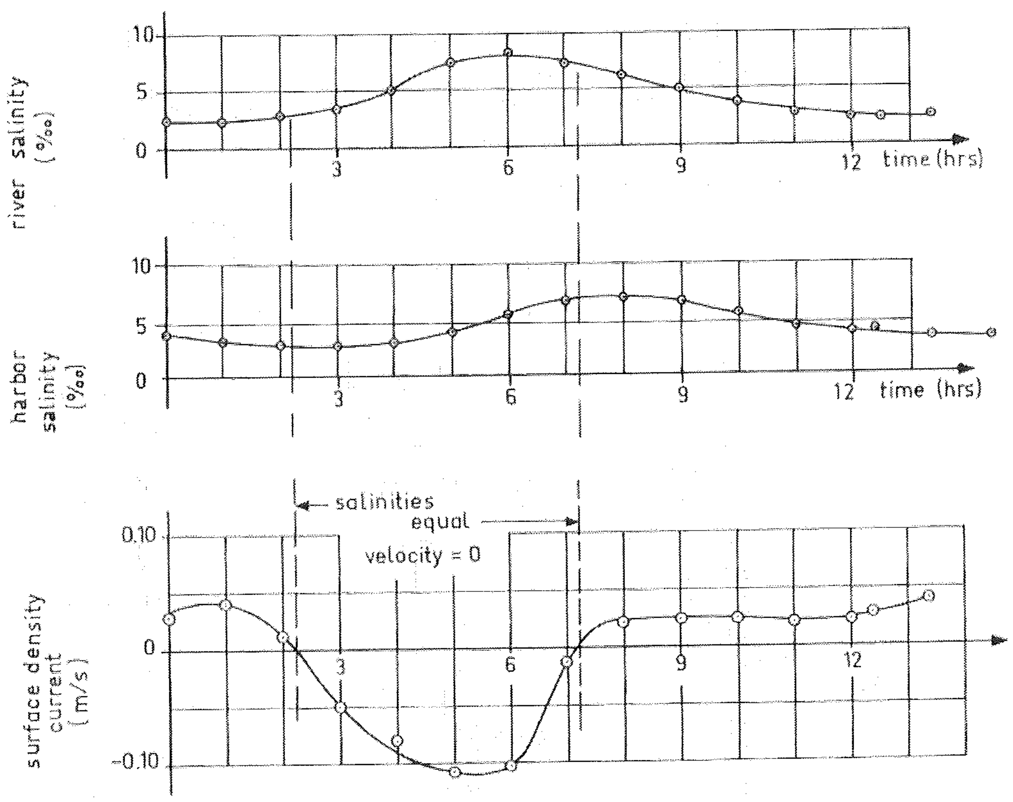


Figure 9-9 Density conditions measured at Rotterdam (mouth 2e Petroleumhaven)

	River	Harbour	δ	V_D at surface
Time	S	S		
(hrs.)	(0/00)	(0/00)	(-)	(cm/s)
0	2.38	3.96	1.224×10^{-3}	+3.0
1	2.47	3.30	5.952×10^{-4}	+4.0
2	2.83	3.04	1.619×10^{-4}	+1.2
3	3.64	2.63	7.830×10^{-4}	- 5.0
4	5.08	3.01	1.600×10^{-3}	- 8.0
5	7.25	3.91	2.567×10^{-3}	-10.7
6	8.06	5.23	2.180×10^{-3}	-10.3
7	7.16	6.56	4.616×10^{-4}	-1.4
8	6.08	6.69	4.679×10^{-4}	+2.1
9	4.90	6.37	1.128×10^{-3}	+2.5
10	3.64	5.43	1.379×10^{-3}	+2.5
11	2.65	4.36	1.325×10^{-3}	+2.1
12	2.38	3.82	1.116×10^{-3}	+2.1

Table 9-2 Density conditions measured at Rotterdam (mouth 2e Petroleumhaven)

Figure 9-8 now shows the idealised current profiles in the mouth of the 2nd Petroleum Harbour based on the combined effect of the filling current and the density current.

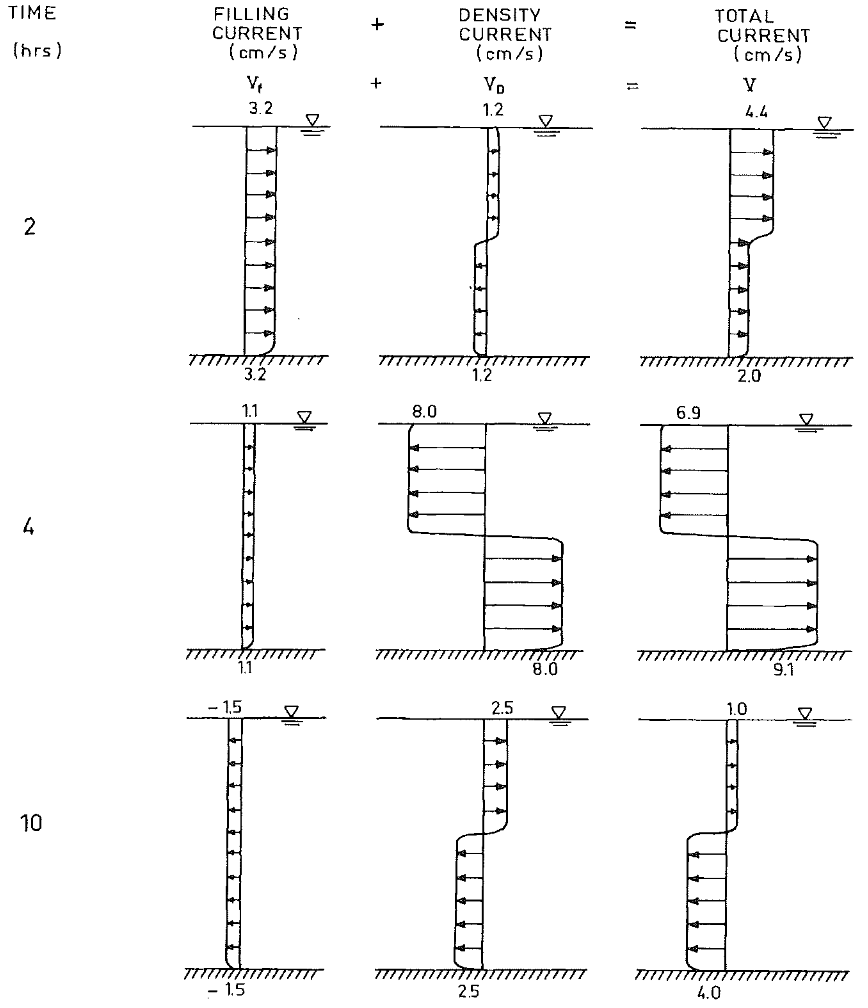


Figure 9-10 Idealised current profiles and their superposition for various times.

In the above approach, it has been assumed that the harbour had an infinite length. In reality this is never true. The average salinity increase in the harbour basin and the density current depends on the configuration of the basin. To determine how far the salinity tongue penetrates into the harbour it is necessary to calculate the continuity of the progression of the density tongue. There are two conditions:

1 The salt must have somewhere to go
2 There must be a driving force (i.e. the density difference)

The first condition is dependent only upon the geometry of the harbour while the second criterion depends on the water alone.

Example. In order to separate these conditions for discussion, let us first assume that initially all the water in a harbour basin and the adjacent river has a density of 1005 kg/m^3. At some instant, the density of the water in the river increases to 1015 kg/m^3, and maintains that value indefinitely; thus, the driving force is maintained. There is no tide. The harbour has a rectangular form and has a depth h = 7 m and length L = 2500 m, Figure 9-11.

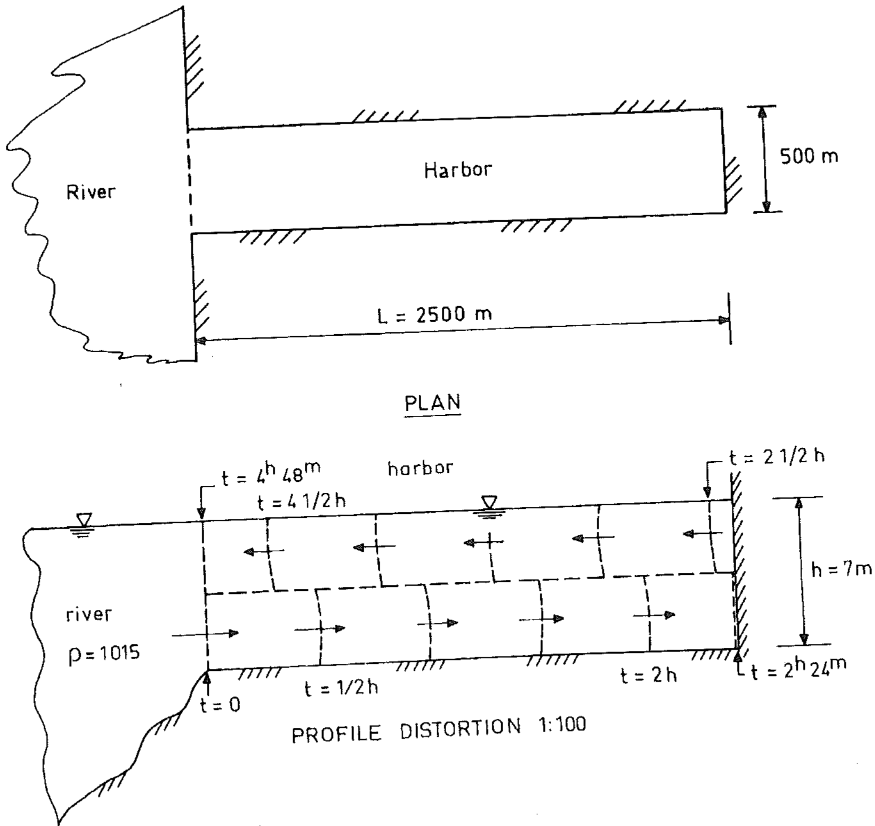


Figure 9-11 Progress of density current in harbour

Using Equation (9.14), with the improved coefficient (0.35 instead of 0.45), we find the density current speed of 1042 m/h. At this speed, the tongue progresses without hindrance over the length of the basin - 2500 m - arriving at the inner end in 2 hrs 24 min. The wave is then reflected from the inner end of the harbour, just as does any other long wave, and propagates back towards the entrance at the same speed arriving there 4 hrs 48 min after the cycle started. The progress of the tongue after each half-hour interval is shown by the dashed lines in Figure 9-11. After the tongue has returned to the harbour entrance, the process stops, since there is no longer a density difference across the harbour entrance.

What has happened to the less dense water that was originally in the harbour? That water has spread over a large area of the river in a thin layer, where wave action enhances its mixing with deeper water. The time required for the density current to enter a harbour and exchange the contents explains the phase lag between the peak salinity in a river and that in an adjacent harbour. Does a complete water exchange take place? It does, except when the driving force (density difference at the entrance of the harbour) is removed in the meantime.

The second type of problem, in which there is insufficient time for a complete exchange, is somewhat more complex. This is illustrated via the following example. This is the same as the previous case in that the harbour initially contains water of $\rho = 1005$ kg/m^3 and the river abruptly changes in density from 1005 to 1015 kg/m^3. This time however, this higher density will be maintained in the river for only 1 hr 12 min, after which the river density will again become 1005 kg/m^3. Indeed, for the first 1 hr 12 min the problem is exactly like the previous example in all respects. After 1 hr 12 min the situation will be as shown in Figure 9-12. The driving force is no longer present. Momentum will keep the slug of salt water moving for a time, but other influences become important, since the trailing end of the slug of dense water is unstable. A dry bed curve will develop at this end of the 3.5 m thick lower layer, causing the slug of water to spread out in a thinner layer along the harbour bottom. Ultimately, of course, this thin layer could retreat entirely to the deeper river. Quantitative evaluations of all these processes are beyond the scope of this course. The complex filling process of a harbour basin, however has major consequences for the sediment transport because large quantities of silt enter the harbour along with the denser water. An impression of the form of the interface between the two water masses some time later is shown in the figure.

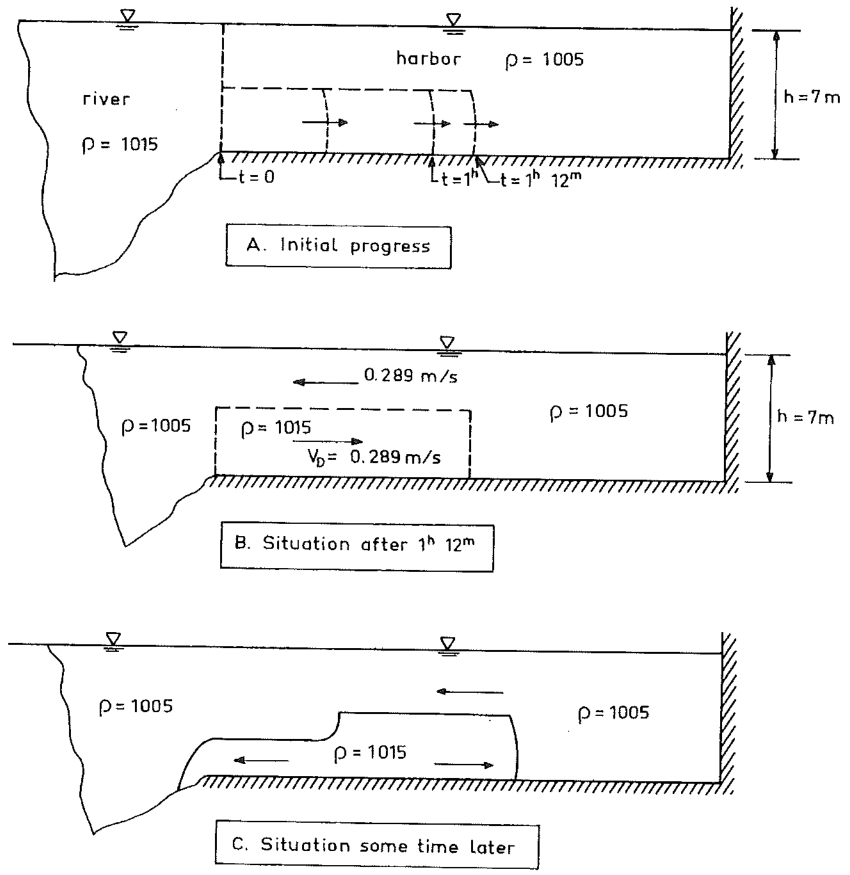


Figure 9-12 Density currents in a harbour

In practice, physical model studies and semi-empirical equations are used to predict maintenance dredging costs and to design measures to reduce the siltation rate. As more than 80% of the harbour siltation is caused by density currents, the water exchange is an important factor. The water exchange depends on the shape of the entrance. Other determining factors for the current pattern are eddies and river currents.

9.4.1 Siltation in harbours

Variations in salinity cause flocculation and the rapid settlement of fine material. This settlement of material proceeds even faster in harbours than in rivers because of the relative tranquillity of the water there. Obviously, all of the phenomena that cause water exchange between harbour and river also increase the supply of sediment to the harbour.

For dredging purposes quantitative information about the harbour siltation is important. This is computed by multiplying the volume of water exchanged in the basin during one tide cycle by the difference between the sediment concentration of in-flowing water and out-flowing water. This is a rough estimate that has practical value. The role of each of the components of the current will be examined in the example in the following section.

9.4.2 The practical problem

The discussion presented above holds true only to the extent that its assumptions are satisfied. The assumption that the river density changes abruptly does not hold true in nature. Moreover, many harbours are not rectangular. Making a dependable theoretical computation of the water exchange in a harbour of arbitrary shape on a given river is extremely time consuming at best. For this reason, physical model studies are often used; WL | Delft Hydraulics is one of the institutions involved in the modelling of saline density currents.

A second approach to the problem is to develop a semi-empirical equation for the water exchange and to determine its coefficients, based upon experience with existing harbours. Such an equation can then be used to predict the exchange taking place in a similar harbour under the same conditions. Since a volume V, can be expressed as a velocity multiplied by a cross-section area and a time, one can expect that the square root of a relative density times a water depth will represent the velocity multiplied by the entrance area A_e, in the equation. The constant tide period and the other coefficients can be combined into one constant. Such an approach has been taken in the Port of Rotterdam. When measurements made in several of the larger harbours there (Botlek, 1^e, 2^e Petroleumhaven) are used the following relationship results:

$$V_d = G A_e \sqrt{\delta' \overline{h}} \tag{9.15}$$

where:

V_d = total water volume passing the harbour mouth in two directions, induced by density current during the entire tide period

A_e = the cross sectional area of the entrance in m^2

G = coefficient depending on the harbour

$\overline{h}$ = average depth of the harbour in meters

δ' = the relative density defined as:

$$\delta' = \frac{\rho_{max} - \rho_{min}}{\overline{\rho}} \tag{9.16}$$

where:

ρ_{min} = minimum river density

ρ_{max} = maximum river density

$\overline{\rho}$ = average river density over one tide period

The method described above depends upon having an existing harbour on the tidal river. Furthermore, the size and geometry of a projected harbour is not always comparable with that of an existing harbour, in such cases, the above scheme is of little help, since the coefficient G cannot be determined. In this case a third option is to use a mathematical model. One should be careful, however, to ensure that the model allows for stratification. A simple 1D model for tidal flow calculations in not good enough.

The density current can be of major importance to harbour siltation resulting in maintenance dredging costs. An estimate of the density current, therefore, can be of vital importance to feasibility studies. Even a crude computation can be helpful in such cases.

The computational technique outlined in this section is illustrated in the example below.

A harbour is located along a river in which the average suspended sediment concentration is 77 mg/l. The harbour is 2000 m long and has a prismatic cross section with side slopes of 1:4. The tide range is 1.7 m and the harbour depth at low water is 13.5 meters.

Figure 9-13 shows such a harbour, with a bottom width of 400 m. The river has a maximum salinity of 8.06 $^0/_{00}$ and a minimum salinity of 2.47 $^0/_{00}$. With a water temperature of 16°C we find that the maximum density in the river is 1005.18 kg/m^3 and the minimum density is 1000.85

kg/m^3, yielding:

$$\delta' = \frac{1005.18 - 1000.85}{1003.02} = 4.32 \times 10^{-3} \tag{9.17}$$

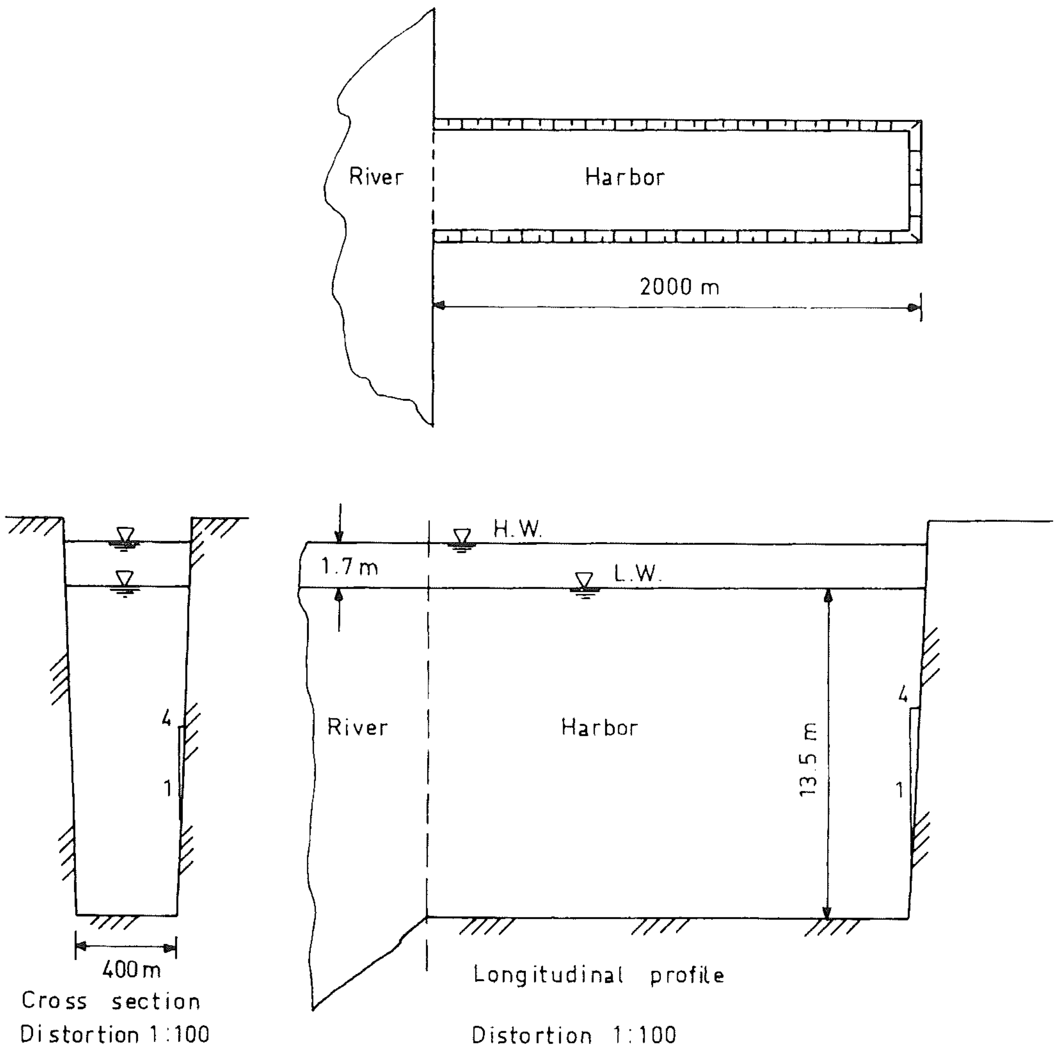


Figure 9-13 Harbour example sketch

The average water depth in the harbour is:

$$\bar{h} = 13.5 + \frac{1}{2} \times 1.7 = 14.35m \tag{9.18}$$

Yielding a top width of:

$$400 + (14.38 \times 8) = 515m \tag{9.19}$$

The average flow area in the entrance is, then:

$$A_e = \left(\frac{1}{2}\right)(400 + 515)(14.35) = 6565m^2 \tag{9.20}$$

The tidal prism, P, of the harbour is the volume of water supplied per tide by the filling current.

$$P=(515)(2000)(1.7)=1.75\times10^{6}\,m^{3} \tag{9.21}$$

Each m^3 of this water carries 77 g of dry sediment into the harbour. Probably not all of this material will settle in the limited retention time. The concentration of sediment in discharged water can be estimated from laboratory tests or from experience in similar local harbours. For this problem, let us assume that the discharge water from the harbour carries an average of 10 mg/l of dry silt. Thus, 67 mg/l (or 67 g/m3) is retained in the harbour.

The amount of sediment transported into the harbour by the filling current (because the density current component dominates the velocity profile, this current is concentrated in the lower layer of the harbour) is then:

$$s_f=(1.75\times10^{6})(67)(10^{-3})=1.17\times10^{5}\,kg\,/\,tide \tag{9.22}$$

The influence of the density current is computed using equation (9.15). The total volume of water exchanged by the density current during a tide period is, using G = 8000 $\sqrt{}$m/tide period:

$$V_d=(8000)(6565)\sqrt{(4.32\times10^{-3})(14.35)}=1.31\times10^{7}\,m^{3}\,/\,tide \tag{9.23}$$

Half of this water, $6.53\text{x}10^7$ m^3/tide, enters along the harbour bottom with the intruding salt tongue and brings sediment S_{d1} with it:

$$S_{d1}=(6.53\times10^{6})(67)(10^{-3})=4.38\times10^{5}\,kg\,/\,tide \tag{9.24}$$

The other half of the exchange water enters the harbour along the surface water as the salt tongue retreats. Since the surface water in a river usually contains less sediment, it transports relatively less sediment into the harbour. For Rotterdam, it is assumed that this surface current transports only 20% of the sediment found in the other currents in the harbour. Since this material will be finer than the average of all the material, it will settle more slowly. Thus, we can still assume that 10 mg/l leaves the harbour alter. These considerations yields:

$$S_{d2}=(6.53\times10^{6})(0.2\times77-10)(10^{-3})=3.53\times10^{4}\,kg\,/\,tide \tag{9.25}$$

The sedimentation from the various sources is compared in Table 9-3. We see that more than 80% of the harbour siltation is caused by the density current.

Component	Quantity (kg/tide)	Percent of total
Filling current	1.17×10^5	19.8
Salt inflow	4.38×10^5	74.2
Salt outflow	3.53×10^4	6.0
Density subtotal	4.73×10^5	80.2
Grand total	5.90×10^5	100.0

Table 9-3 Harbour sedimentation summary

A very practical question remains for those responsible for the maintenance of the harbour. How much shallower will the harbour become as a result of siltation over the course of one year? This can be answered if the densities of the dry sediment particles and of the in situ sediment are known. Reasonable values for these are 2650 kg/m^3 and 1200 kg/m^3 respectively. Then, if v_v denotes the volume of water-filled voids in 1 m^3 of sediment:

$$1200=(2650)(1-v_v)+(1000)(v_v) \qquad (9.26)$$

from which $v_v = 0.88$. Therefore, 1m^3 of sediment contains:

$$(1-0.88)(2650)=318\,kg \qquad (9.27)$$

of dry sediment particles. 5.9 x 10^5 kg of sediment particles occupies a volume of:

$$\frac{(5.90\times10^5)}{(318)}=1855\,m^3 \qquad (9.28)$$

This volume of sediment accumulates in one tide period. There are:

$$\frac{(365.25)(24)}{(12.42)}=706 \qquad (9.29)$$

tides per year, so that in one year, the accumulation of sediment in the harbour is:

$$(1855)(706) \ = \ 1.31\times10^6 m^3\,/\,year\,! \qquad (9.30)$$

This volume is spread over the harbour bottom in a layer thickness of :

$$\frac{(1.31\times10^6)}{(2000)(400)} \ = \ 1.64\,m \qquad (9.31)$$

It is usually not economical to dredge out a sediment layer less than about 2.5 m thick. In this case the harbour could be dredged about once every 18 months.

This last dramatises the importance of the density current. If the density current could be eliminated in the harbour, then the interval between dredging activity could be increased by about a factor 5 (see Table 9-1) or to about 7 years. The economic savings involved are obvious.

The crude computerisation will not always yield reliable results since the current pattern in a harbour mouth can be complicated. The complication can exist in the form of an eddy rotating about a vertical axis in the harbour entrance. Water exchange between the harbour and eddy on one side and between river ad eddy on the other can increase the transport of salt and suspended sediment into the harbour. When a harbour is small, the density current can usually carry out a complete water exchange rather quickly, but then stops transporting silt-laden water into the harbour. The eddy, on the other hand, continues functioning, exchanging sediment laden river water for clearer harbour water. This can be the most important of the three causes of the transport of sediment into a small harbour.

Eddies form at the entrance to larger harbour basins as well. However, these tend to be excited by the other current components in the harbour entrance rather than the river current. As such, they contribute little to the supply of sediment to the harbour. It takes little imagination to realise that near the mouth of a harbour, where eddies, density currents, river currents and harbour-filling currents are all competing with one another, the current pattern can be rather confused. Small, shallow draft ships will only be concerned with the surface currents. Larger ships with a deeper draft, which penetrate the interface between layers, are subjected to dead water phenomenon and we should realise just how much respect is due to the pilots who cope with these uncertain conditions.

9.4.3 Methods to combat density currents in harbours

Combating density currents happens via:

1 Narrowing the entrance; as the volume of water exchanged is thought to be proportional to the entrance area. Thus, reducing the entrance width should reduce the volume of exchanged water. In practice, such a narrowing will not be quite as effective as desired. The intruding density current stream will spread in both horizontal directions in the wider harbour basin; this tends to increase the effective driving force by increasing the slope of the interface between the water masses. This effect is difficult to quantify, however.

2 Installing a single set of gates at the entrance to the harbour. The harbour level is then maintained at a constant level - even the filling current is eliminated. The water level in the harbour remains constant; this is convenient for the cargo handling operations. Does a density current cause a water exchange? Not necessarily. If the gates are opened only once during the tide cycle and at the time in the cycle when the water levels are equal, then the harbour water will eventually have the same density as the river water and no dredging problems will be experienced. On the other hand this means that the doors are opened only once in every tidal cycle and it may be unacceptable to force the shipping to wait so long to pass through the entrance. What would happen if the gates were opened twice per tidal cycle while the water levels were the same - once on a rising tide and once on a falling tide? There would still be no filling current, but there is no guarantee that the density in the river would be the same at both times. Usually it will not be, and a density current and water exchange will take place during the time that the gates are open. Indeed, such a solution is of little value except when very great variations in tide level might make cargo handling inefficient in an open basin. This method is not effective if the sediment concentrations are very high. In that case the suspended sediment itself has a dominating influence on the density.

3 Putting in a lock: depending on the depth of the tidal navigation channel ships might be able to enter and leave the harbour at any time irrespective of the water levels. Each locking operation can be accompanied by a water exchange within the lock. Since the lock is relatively small, this exchange progresses rather rapidly - 27 minutes for the large lock at IJmuiden, for example. Special facilities have been built at IJmuiden to trap this intruding salt water and retain it for later disposal. These special facilities consist of a deep pit just landward of the lock connected via an equally deep channel to a sluice.

4 Constructing a deep pit connected to a sluice: salt water coming in through the inner gate of the lock falls into the pit. Later during low tide at sea this salt water can be discharged through the sluice.

5 Using an air-bubble curtain: this is a stream of rising air bubbles released from a perforated submerged pipeline at the end of the lock near the door. The rising bubbles increase the turbulence and hence the mixing; this reduces the driving force of the density tongue and reduces the intrusion.

A combination of measures can be taken. Other more unusual devices have been proposed from time to time to combat density current intrusions into harbours. For example, a device that looks like a giant brush with vertical bristles that bend in order to allow a ship to pass. Many other similar devices can be conceived by using a bit of ingenuity.

10. PRACTICAL PROBLEMS AND COMMON SOLUTION METHODS

10.1 Introduction

Large parts of sandy coasts all over the world suffer from structural erosion and/or dune and beach erosion during severe storm surges. In coastal engineering practice an important aim is the proper protection of these threatened coasts. Besides this type of protection, sometimes newly reclaimed areas have to be protected from the attacks by the sea.

From the many available methods, coastal engineers have to select a proper tool. Protection with the help of 'hard' structures is often used. Series of groynes, series of offshore breakwaters, submerged breakwaters and revetments or seawalls are examples of 'hard' structures. With the help of these 'hard' methods, it is possible to interfere in the actual sediment transports in cross-shore and longshore directions of the coasts. In addition, there are 'soft' methods like dune, beach and shore face nourishment. With the help of these 'soft' methods, the principle is to avoid interfering in the natural sediment transport processes. With this system, only the losses of sediment occurring in a stretch of coast are to be compensated on a regular basis.

Whether to select a 'hard' or a 'soft' method depends on the characteristics of the problem concerned and on economic considerations. In Coastal Zone Management practice, the use of beach nourishment is increasingly popular. Many of the frequently occurring adverse side effects of 'hard' structures can be avoided by using artificial nourishment.

However, the possible use of hard structures for coastal protection still cannot be disregarded. A coastal engineer should at least have proper insight into the physical properties of 'hard' structures. The desired effects (often: reduction or mitigation of the erosion potential in a given stretch of coast), and the unwanted, often detrimental effects on adjacent coasts, have to be considered with care. Only then can an appropriate choice be made between the many coastal protection methods.

10.2 Coastal protection problems

Various problems in coastal engineering practice call for appropriate countermeasures. Some of these will be first briefly discussed; in the next section the use of structures to solve problems will be outlined. The restricted number of problems discussed are:
- structural erosion of coasts
- beach and dune erosion during severe storm surges
- the protection of newly reclaimed areas
- the stabilization of dynamic tidal inlets

10.2.1 Structural erosion of coasts

In fact, *coastal erosion* is a rather tricky notion. It is beyond doubt that the erosion of the sandy coast, which is often observed at the leeward side of port entrances sheltered by two breakwaters, is a typical example of a structural erosion problem (see Figure 10-1). Structural erosion also occurs as a result of long-term adaptation of the position of the coastline in an area where for example, because of up-stream damming of the river, the sediment input from the river has been reduced.

In the erosion area the volume of sand in an arbitrary cross-section (m^3/m) between well-chosen boundaries in that cross-section, are apparently gradually reduced as a function of time. Year after year, such a volume is reduced. Typical orders of magnitude of this type of erosion are 10 to 50 m^3/m *per year*. Structural erosion is a *permanent* erosion phenomenon. The construction of a harbour on a sandy coast and the up-stream damming of a river are clearly man-made actions with adverse effects on coasts. Purely natural reasons can also be associated with some structural erosion problems.

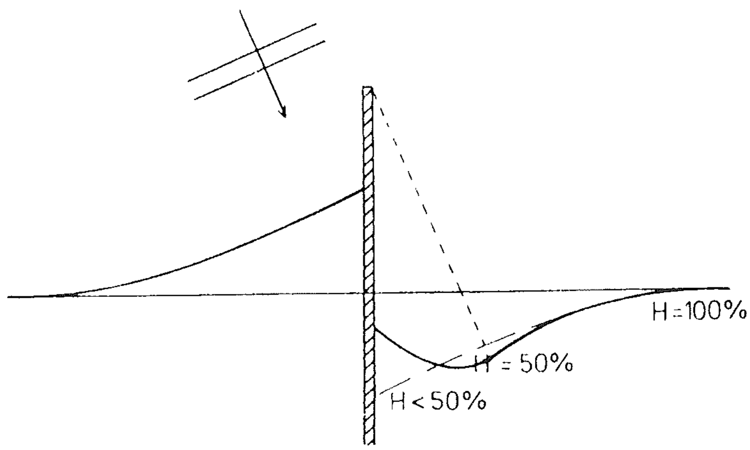


Figure 10-1 Typical structural erosion problem on the lee-side of a breakwater

The erosion of the beach and dunes or, in the absence of dunes, direct erosion of the land during a severe storm surge (see next section) is also considered as a typical erosion problem. Indeed, after the storm event the dunes and/or upper parts of the beaches may have lost sediment, which has disappeared from its pre-storm position. Often, however, the lost volume is found on the foreshore in the nearshore area. (See Figure 10-2) Essentially, the total volume of sand in a cross-shore profile has not changed during the storm surge. In fact, only a redistribution of the sediment masses over the cross-sectional area has taken place. Depending of course on the severity of the storm surge, the volume of sand lost from the upper parts of the cross-section associated with this type of erosion is in the order of magnitude of 10 to 100 m^3/m per event (say: *per day*). In the period after the storm surge event, the ordinary natural conditions often force a recovery of the beaches and dunes. Dune erosion during a severe storm surge is a *temporary* erosion phenomenon.

In a typical structural erosion problem, both normal conditions and storm conditions contribute to the eventual loss of sediment from of a cross-section. Often a *gradient* in the longshore sediment transport is the main reason. [Gradient: $dS/dx \neq 0$; S: net yearly longshore sediment transport; x: coordinate directed along the coastline.]

Structural erosion under normal conditions entails that the upper part of the profile (dry beach and slope of dune or mainland front) does not participate in the transport processes; the water and the waves do not reach this part of the cross-section. However, the erosion of the foreshore will continue under these conditions. Only if the waves reach the dunes (under storm conditions, higher water level and higher waves), does the upper part of the cross-section form an integrated part of the entire active profile. In that case, the erosion of dunes or firm land will occur. While in a basically stable situation this erosion of dunes or firm land is only temporary, in a structural eroding case this erosion is partly permanent. During normal conditions, sediment from the upper part of the cross-section will not fully return, but will be removed in the longshore direction. At the end of the day, with the 'help' of cross-shore transport processes gradual erosion will also cause erosion of the dunes or firm land. This distinction is often not clear to outsiders. Ultimate permanent losses of dunes and firm land are incorrectly directly associated with storm surge events, while the basic problem is still the structural erosion problem.

Structurally eroding coasts are often the source of serious problems for the various users of the coastal zone. Properties built close to the sea are ultimately lost; roads in the area disappear into the sea. Often society calls for action to be taken by the Coastal Zone Authorities in order to prevent the detrimental effects of structural erosion.

10.2.2 Beach and dune erosion during severe storm surges

Coasts which, over a number of years, seem to be stable, may suffer from the effects of storm surge events. It can be argued that during storm surge conditions the shape of the pre-storm profile is far out of the equilibrium shape which belongs to the severe storm surge conditions. Often a temporary increase of the still water level (surge) and far higher waves than those experienced in normal conditions are associated with a storm. Erosion of the upper part of the cross-shore profile, while the foreshore is accreting results in flatter slopes of the profile during the storm. (See Figure 10-2) While over time the profile is flattening, the erosion process slows down.

As mentioned in the previous section, the rate of erosion can be very high (of course depending on the actual conditions). Associated with the loss of volumes of sediment from dune or firm land, a retreat R of the coast (see Figure 10-2) of many meters may occur during a single event. Nowadays methods are available to reliably quantify the rate of dune erosion during arbitrary boundary conditions (see Chapter 5).

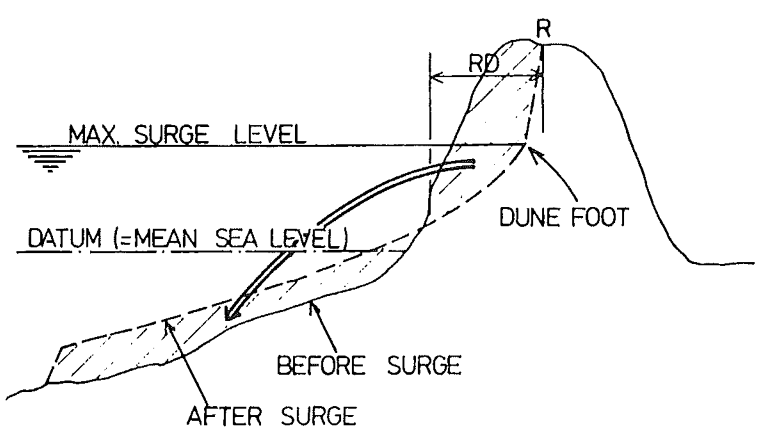


Figure 10-2 Dune erosion during a severe storm surge

The effects of permanent structural erosion or of temporary erosion because of a storm surge, on properties built too close to the shoreline are eventually the same. In both cases, the properties may be lost. (See Figure 10-3) However, it is beyond doubt that countermeasures meant to resolve both these types of erosion must be quite different.

Figure 10-3 Damage because of dune erosion

10.2.3 Protection of newly reclaimed areas

Here the coastal zone is simply defined as consisting of the beach, an adjacent strip of sea and an adjacent strip of land parallel to the beach. The width of the latter strip may be several kilometers. All over the world such coastal zones serve many different important functions. Sometimes there is a shortage this type of useful area. When trying to acquire more space the reclamation of land from the sea may be considered as an option. An expansion of the land may be achieved by shifting the coastline in a seaward direction with the help of huge artificial sediment depositions. Artificial islands are sometimes made. In both cases, structures can be used to form the boundaries of the reclaimed area. Options include structures similar to a full dike or revetment protecting the core of the reclaimed area, or some fixed points between which protected sandy beaches may develop.

10.2.4 Stabilization of dynamic tidal inlets

The position of the main channels of tidal inlets is often not fixed. Natural conditions may cause a gradual shift in a specific direction; however sometimes the position of the main channel moves back and forth. Mobile behaviour of a tidal inlet is often detrimental to safe navigation and the integrity of properties built on both adjacent land areas may become endangered. Sometimes the mobile behaviour of a tidal inlet system is considered undesirable. Stabilization of the inlet system is then required. Structures assist in securing this aim.

10.2.5 Discussion of coastal protection problems

A few general coastal engineering problems have been briefly discussed in the preceding sections. Different problems call for quite different solutions. Each problem is caused by specific conditions. In developing proper solutions to each specific problem, one has to meet problem-specific requirements. This calls for proper insight into the physics of the problem. Sediment transport plays an important role in all problems discussed so far. In solutions using structures ('hard' solutions), these structures must subtly interfere in the sediment transport involved. In the next section, this topic will be given further consideration.

10.3 Use of structures in coastal protection

In most cases the use of structures as a tool for coastal protection relies on the ability of such structures to interfere with the existing sediment transport processes. For stretches of coast suffering from structural erosion, often a *gradient* in the net longshore sediment transport along that stretch is the main reason for the erosion problem. Figure 10-4 shows a plan view of such a stretch of coast (lower part). In the upper part of Figure 10-4 the net yearly sediment transports S along the coast (S: expressed in m^3/year) have been indicated schematically. Apparently the sediment transport increases with increasing distance x along the coast. Line *a* of Figure 10-4 represents this longshore sediment transport distribution. The increasing transport from A to B (difference V) causes the erosion problem in stretch A-B. (Along A-B $dS/dx \neq 0$.)

Assume that one wishes to protect only stretch A-B of the coast (for instance because important investments have been made in section A-B), and these are at stake owing to the structural erosion processes. The necessary and sufficient action for that goal is to ensure that the existing sediment transport distribution line *a* in Figure 10-4 should be changed to line *b*. (along section A-B, $dS/dx = 0$ in that case.) At least in section A-B the erosion would stop if distribution *b* could be achieved. In the left-hand section from A the erosion will just continue; the sediment transports have not changed. In the right-hand section from B the existing erosion continues; at an even higher rate since the throughput of sediment through the cross-section in point B has been reduced, yielding steep gradients in the longshore sediment transport distribution. At the right-hand side of B consequently *lee-side* erosion occurs.

The formulation of the requirements of the sediment transport distribution in section A-B is rather simple; however, it is rather difficult to acquire curve *b*. Structures that, in principle, interfere with the process of longshore sediment transport rates (longshore drift) can be used. Series of groynes, series of offshore breakwaters or series of breakwaters with a crest either above (emergent) or below sea level (submerged) will undoubtedly affect the existing longshore sediment transport, but it is very difficult to ensure that these countermeasures are appropriate for specific cases. If the effectiveness of the countermeasure does not come up to expectations, the erosion will be reduced but not entirely stopped. If the design is too effective (the reduction of the sediment transport in stretch A-B is too great, see line *c* in Figure 10-4), accretion in stretch A-B will be unavoidable. This might be beneficial for section A-B (accretion instead of the desired stabilization), but the lee-side erosion in the section at the right-hand side of B will grow worse. In fact, achieving line *c* in Figure 10-4 represents an 'over-kill' operation.

Even if the countermeasures in section A-B are well tuned, lee-side erosion beyond B will occur. Often eventually, this increased erosion beyond B also becomes unacceptable, calling for also countermeasures in this section. The lee-side erosion is then shifted further down the coast.

Lee-side erosion is usually unavoidable if an effective design is made for section A-B. In fact the solution of the erosion problem in section A-B with the help of structures that decrease the sediment transport along A-B comes at the expense of the stretch of coast beyond B; the problem has simply been *shifted*. Only if there is an accreting stretch of coast beyond B, or if position B represents the very end of a stretch of coast (e.g. a tidal inlet beyond B) is lee-side erosion contained/less obvious.

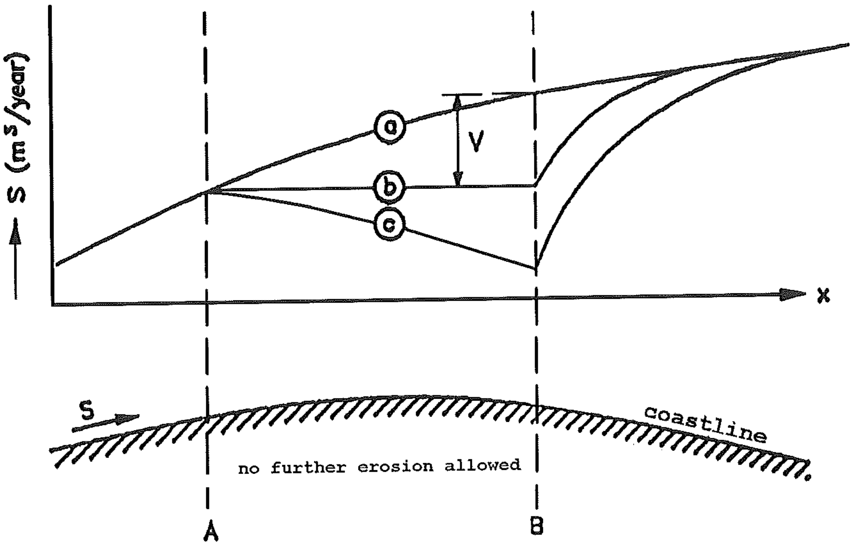


Figure 10-4 Longshore sediment transport distribution along eroding coast

If artificial nourishment has been selected as countermeasure in section A-B, volume *V* (see Figure 10-4) must be nourished on a regular basis. In practice, it is not useful to nourish *V* every year, so usually a lifetime of 5 to 10 years is chosen for a nourishment.

The erosion problem is not solved by artificial nourishment; the nourishment does not reduce the sediment transport involved. The erosion continues so consequently the nourishment has to be repeated. That seems to be a drawback. In many cases, however, artificial nourishment is more cost-effective than solutions with structures. This is certainly the case, when the present day value of the measures is calculated (future nourishment contributes little to the present day cost). An additional advantage of artificial nourishment is that lee-side erosion does not take place.

Many details related to the use of artificial nourishment are found in Coastal Engineering's Special Issue on Artificial Beach Nourishment (1991). That Issue contains many references to papers specially devoted to artificial nourishment.

In this chapter the use of structures is the main point of interest. In this introduction the structural erosion problem indicated by Figure 10-4 was chosen to clarify the possible use of structures in solving the erosion problem. Interference with existing sediment transport had become a necessary requirement of structures as countermeasure. From this requirement, it can easily be understood that in principle series of groynes, series of detached offshore breakwaters and submerged breakwaters can be used as a tool. All these possibilities are able to affect the existing longshore sediment transport.

Series of groynes: Groynes (built more or less perpendicular to the coast) may reduce the wave driven longshore currents directly. Possible tidal currents parallel to the shore are diverted to deeper water. Both effects contribute to the desired decrease in the longshore sediment transport.

Series of detached offshore breakwaters: Without going into detail, it can be argued that series of detached offshore breakwaters built at some distance seaward of the shoreline, are able to reduce the wave heights in the zone landward of the structures. Reduced wave heights yield a reduced longshore sediment transport.

Submerged breakwaters: Even submerged breakwaters (crest height below Mean Sea Level and built parallel to the shore) are able to reduce the wave heights in the zone landward of these structures. In this case also a reduction of the longshore sediment transport rate may be expected. One should be aware of the negative effect of rip currents through the openings between the breakwaters.

The use of **seawalls or revetments parallel to the shore**, built along the front slope of the dunes or the firm land, has on purpose **not** been mentioned as a possible countermeasure to the structural erosion problem in stretch A-B of Figure 10-4. It is a very bad solution to this type of erosion problems.

Just after the construction of these structures, the loss of dunes or land is indeed effectively prevented, but the cause of the underlying erosion problem is not cured. The erosion of the beach and nearshore will continue because of the existing gradient in the longshore sediment transport. Later, the attack on the structures will increase owing to the loss of beaches. Damage will occur; the structures have to be strengthened. Finally, the beaches in front of the seawalls or revetments have disappeared and only the heavy structures remain to protect the land (see Figure 10-5).

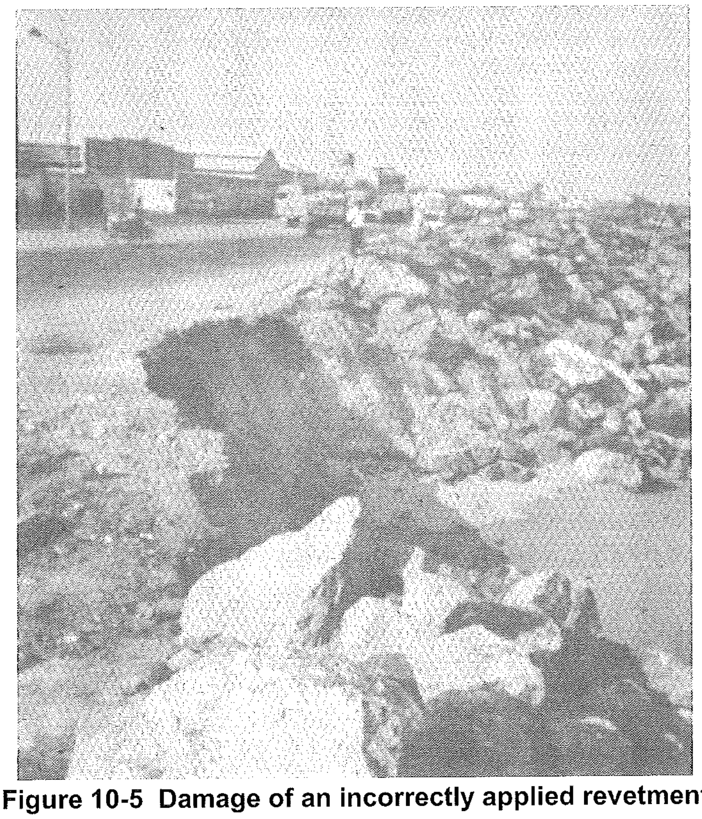


Figure 10-5 Damage of an incorrectly applied revetment

To finish this introduction a few remarks can be made:

- From the erosion example of Figure 10-4 it has become clear that the solution of a coastal erosion problem always starts with a clear definition of the problem. This holds for the structural erosion problem, and similarly for all other coastal engineering problems.
- A clear definition of the requirements for a possible solution has to be given. Should the erosion be prevented along the entire coast or only in a limited area? Is halting the erosion in a limited area enough or is the recovery (accretion) in that area desirable?
- In the next phase, different alternatives have to be analyzed. Which alternatives meet the requirements? What are possible undesirable detrimental effects side effects? What are the costs involved? In the final selection phase the best' alternative has to be chosen.
- After implementation of the selected alternative in the field, it is strongly recommended that the actual behaviour of the coast to be protected should be monitored. Because 'art' and 'science' are still closely related in coastal engineering practice, it is possible that in some cases

it has to be concluded that in fact the chosen solution was far from ideal. The experience gained in this way can be very helpful while designing new projects.

- Coastal protection as part of coastal engineering practice is a difficult task. Skilled and experienced professionals are required to do the job. In some countries special Institutes or authorities have been appointed to carry out the tasks involved. (Coastal Zone Authorities.) It is obvious that such Authorities can only adequately operate if provided with governmental support and legal backing.

10.4 Solutions with structures to problems as mentioned

In the present section some specific solutions using structures to the basic problems briefly discussed in the preceding paragraph are given. These structures are often referred to as hard solutions, in contrast with the soft solutions that were discussed in section 10.5. The hard solutions have in common that they are permanent structures and that it is difficult to modify them if they do not function as expected. From an economic point of view, it is important to realise that a large up-front investment is required.

The functionality of the different solutions with respect to the various problems will be the point of interest. In all cases it is assumed that stable structures have been designed. Appropriate structural design topics are not addressed. A recent overview of these topics can be found in 'River, Coastal and Shoreline Protection' (1995) and in the 'Shore Protection Manual', CERC (1984).

10.4.1 Structural erosion of coasts

Some aspects of the structural erosion of coasts have already been discussed as a specific example in the Introduction of this section. Structures may be used as an alternative solution to the structural erosion problem.

In principle, series of groynes (or as an alternative: rows of wooden or steel piles) are able to reduce existing sediment transport. The fine-tuning problem (length and length/mutual spacing ratio) is, however, a difficult problem to resolve. No generally applicable design rules are available although there are some rules of thumb. In the example case of Figure 10-4 in the cross-section through point B the existing sediment transport had to be reduced by an estimated factor of 0.5 in order to fulfil the requirements. (I.e. achieving line *b* in Figure 10-4; $S_{B\,new} \approx 0.5\ S_{B\,old}$.) Notice that for arbitrary cross-sections between A and B different (larger) ratio's are required. Let point C be half way between A and B; then according to Figure 10-4: $S_{C\,new} \approx 0.7\ S_{C\,old}$.

Another point of practical concern is the absolute magnitude of the net yearly longshore sediment transport involved. In Figure 10-4 the basic erosion problem for stretch A-B is the loss of volume V out of that stretch. The volume in question can be determined by monitoring the coast for some time. Although the *difference* V of S_B and S_A can be measured quite accurately but the absolute values of the net yearly sediment transport of either S_B or S_A are in fact not automatically known. Since it is very difficult to calculate these sediment transport rates, easily errors can be made in the proper quantification of S_B and S_A. If in Figure 10-4 both S_A and S_B are increased by ΔS, the difference V remains the same. However, in this case also in order to achieve a constant sediment transport in section A-B quite different reduction factors from those mentioned previously are necessary.

If the erosion problem of stretch A-B has been resolved by installing groynes, extra erosion with volume V in the area just at the right-hand side of point B has to be taken into account (lee-side erosion).

With the help of series of emergent shore-parallel offshore breakwaters in section A-B, the erosion problem of stretch A-B can be resolved. In principle such breakwaters are able to interfere in the longshore sediment transport (are able to reduce this transport). Due to the partial shadow effects of the breakwaters, the general wave conditions landward of the series breakwaters are reduced, yielding less sediment transports. However, the fine-tuning of series of offshore breakwaters is a difficult task. In the example of Figure 10-4 the sediment transport in the cross-section through point B must be reduced from S_B to S_A. Therefore, near section B, the remaining longshore transport should not be 0. Either in the area landward of an offshore breakwater near B or in the area seaward of that breakwater on-going sediment transport is still required. In the first case, this calls for the forming of a feature called a salient; in the latter case a tombolo may form.
With a proper series of offshore breakwaters the lee-side erosion at the right-hand side of B is also unavoidable.

A continuous submerged breakwater parallel to and at some distance from it will undoubtedly reduce the wave heights landward of the submerged breakwater (depending on the wave climate, the location and the crest height relative to the still water level). By reducing the wave heights, a reduction of the sediment transport can be expected. This holds for both cases with only wave driven longshore sediment transport and for cases where a combination of wave driven currents and tidal currents occurs. A first approximation suggests that in the latter cases the tidal currents landward of the submerged breakwater are unaffected. However, the resulting sediment transport will be greatly reduced by the reduction of the wave heights in the area landward of the submerged breakwater.

If an infinitely long submerged breakwater is considered, no special additional problems are expected. In practice, however, infinitely long submerged breakwaters are not a realistic option. If, for example, as in Figure 10-4 in stretch A-B, only local provisions are required, at least two end sections of a submerged breakwater occur. In association with the partial breaking of the waves above the crest of the breakwater, a mass transport of water over the breakwater takes place. In a two-dimensional case (i.e. in a wave flume in a hydraulic laboratory), this mass transport is compensated in the same cross-section. However, in a three-dimensional case the landward-directed mass transport is often collected behind the submerged breakwater in an ever-increasing shore parallel current. Along the end sections of the breakwater this current escapes in the seaward direction like a rip current. Examples are known from field and from laboratory tests in which severe erosion is generated landward of the submerged breakwaters.

As already explained, constructing seawalls parallel to the shore or revetments along the front slope of the dunes or mainland does not provide an adequate solution to a structural erosion problem .

10.4.2 Beach and dune erosion during severe storm surges

If the rate of erosion due to a severe storm surge is felt to be too large during design conditions, the use of structures might be helpful in reducing the rate of erosion. Series of groynes do not 'work' to reduce the associated offshore directed sediment transport. In principle, series of emerging breakwaters or submerged breakwaters are able to reduce the wave heights landward of these structures and consequently may have the effect of reducing the rate of dune erosion. However, often an increase of the still water level is associated with a storm (the surge); consequently the effectiveness of these type of structures in reducing the wave heights is limited. Besides the use of these structures has large undesirable implications relating to the longshore processes.

If a basically stable (stable: with respect to structural erosion) situation is considered, the use of seawalls or revetments may be useful to restrict the rate of erosion during a severe storm surge. These structures physically prevent the loss of material from the dunes or land. The desired reduction in the retreat of the dunes or mainland can be achieved. Since *behind* the seawalls or revetments no material can be eroded to provide the offshore directed transport capacities, one has to expect a lot of erosion just *in front* of the structures. A deep scour hole can be anticipated; this potential scouring has to be taken properly into account in the design of the structures. Steetzel (1993) proposes a method to calculate the scour depth in front of seawalls and revetments.

10.4.3 Protection of newly reclaimed areas

Assume a coast that has to be extended for a considerable distance (say order of magnitude 2 km in cross-shore direction and over 20 km in the longshore direction). It is assumed that no serious erosion problems occur the existing situation. The area is intensively used as beach recreation area. One of the requirements is that after extension of the coast, recreation beaches are again available. To protect the newly reclaimed area simply by a dike or revetment is consequently not an acceptable option.

A more-or-less zero option for land reclamation would be an entire shift of the cross-shore profile of 2 km in seaward direction. This holds not only for the waterline, but also in principle also for all other depth contours to a water depth where natural adaptations of the profile are hardly to be expected. In some cases, this depth has to be estimated at say 15 m below Mean Sea Level (MSL). In order to achieve a 2 km shift of the coastline, per running meter along the shore 2000 m x 20 m = 40,000 m^3/m is required. (The factor 20 m in the calculation is found by assuming a lower limit of MSL -15 m and an upper limit above MSL of 5 m.) With a longshore extension over 20 km, the total volume of sediment supply reaches a volume of 800 million m^3. This is a really huge project!

With the zero option the new coast hasa foundation that is identical to the old coast. A large part of the calculated volume is needed to make the new foundation. In order to restrict the volume of sediment needed for reclamation in the zero option, one could consider an alternative. For example, the upper part of the cross-shore profile after reclamation is 'supported' with the help of a submerged breakwater. The toe of the profile can then be avoided; a large reduction of the volume of sand is achieved in this way. This solution is called a "hanging beach".

The seaward limit of the reclaimed area can also be moulded with series of detached breakwaters.Many, not yet totally resolved coastal engineering, problems have to be faced before adequate structural designs can be made. This holds especially for the three limits of the reclaimed area that are bounded by water.

10.4.4 Stabilization of dynamic tidal inlets

Much experience has been gained in the past in resolving the problem of stabilization of dynamic tidal inlets. In most cases long groynes or jetties have been constructed on both sides of the inlet. Since in most cases the natural sediment transport over the ebb tidal delta is fully interrupted, a proper design of the total solution calls for additional provisions in order to restore the sediment transport across the tidal inlet. A sand by-pass system should be an essential part of the design. Estimation of the required capacity of a sand by-pass system is a very difficult task. Often that part of the total design is more-or-less postponed until nature shows by accumulation at one side and by erosion at the other side, the real quantities involved. By postponing (wait and see) it sometimes emerges later that there is no money left to build a proper sand by-pass system.

10.5 Solutions without structures to problems as mentioned

In this section how problems of coastal erosion can be solved without building structures of quarry stone or concrete is considered. These types of solutions are called soft solutions. The basic idea is to supplement sand by artificial means in places where the loss of sand is causing problems. The method works if ample quantities of sand are available at a short distance from the problem area. Since erosion is an ongoing process, the nourishment will have to be repeated from time to time. The interval between successive supply operations depends on the rate of erosion and on the cost of mobilising the dredging equipment. Generally an interval of five years between successive operations is considered acceptable. The distinction between the hard and the soft methods is that the latter must be repeated from time to time, that it is flexible (in the sense that it is easy to modify the scheme if the results are not as expected) and that the cost of the operation is deferred in the sense that it is spread over a longer time. For conditions along the Dutch coast this makes the soft solutions more economical than the hard solutions.

10.5.1 Quality requirements for the sand

When sand is supplied to an eroding stretch of coast, the newly applied sand will tend to form a crust or blanket over the existing coastal formations. The newly applied sand will follow the physical laws that dominate the morphology in the same way as the existing sand, For this reason major changes in the slopes and other coastal features are to be expected when the grain size of the supplied sand differs from the original material. Usually, this is not acceptable, and one of the basic rules for beach nourishment is the use of material that is similar to the existing material.

Another point of concern is the silt content of the suppletion material. Since the bottom material of a sandy shore is extremely mobile, specifically in the breaker zone, any fines will have been washed out long ago, so that the water is relatively clear. This is an essential condition for biological processes and for the ecological equilibrium of the coastal zone. It must therefore be ascertained that the beach nourishment material contains little or no fines. In the worst case, measures have to be taken to wash out the fines before placing the material on the beach.

10.5.2 Origin of the sand

The sand used for beach nourishment can be obtained from land based sources or from marine sources. The land-based sources may be riverbeds or dry sand deposits. The marine sources can be estuaries or the seabed. It is often attractive to try to combine necessary excavation works that are with nourishment activities, to reduce the overall cost. (In Dutch: werk met werk maken).

When marine material is used it must be dredged at a sufficient distance from the shore to prevent extra erosion due to the presence of the borrow pits. Specifically, when material is dredged from the seabed, the question arises of whether it is better to dredge the material from small extremely deep borrow pits, or to dredge thin layers of the material from a very extended borrow area. Dredging thin layers means disturbing the biologically active surface layer in a large area, whereas creating deep borrow pits enhances the risk of stagnant water of poor quality remaining in the deeper parts. Although research into the effects of sand winning is being carried out, there is no clear conclusion yet. In the Netherlands, some sand is obtained by maintenance dredging in the access channels to Rotterdam and IJmuiden. The remaining amount of sand that is required is extracted by dredging thin layers at a distance of at least 20 km from the shore.

10.5.3 Places where suppletion is used

Sand supplied to the coastal system can be placed at different locations in the cross section. The decision depends on the purpose of the nourishment and sometimes on the source of the material. The basic choices are (see Figure 10-6):

1. On the inner slope of the dunes
2. On the outer slope of the dunes
3. On the dry beach
4. On the foreshore

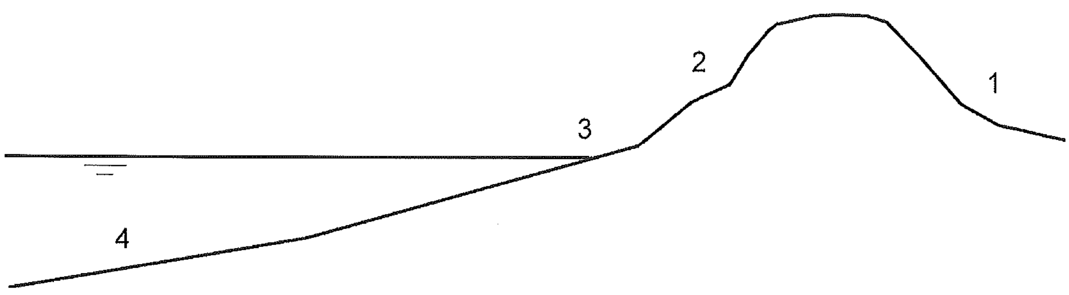


Figure 10-6 Locations of sand suppletion

Material is usually placed on the dunes in cases where calculations have shown that the volume of material in the dune ridge is insufficient to cope with dune erosion during the design storm. In such cases, the hinterland may be exposed to flooding, not because of ongoing erosion, but simply because the existing dunes form too small a stockpile to cope with extreme conditions. The use of sand from land based sources is a reasonable option in these cases. This is even more important if the use of marine sand would cause a salt intrusion problem in a vulnerable region.

If the nourishment is required to counteract ongoing erosion, the most common place of application is the dry beach. Sand is placed between the LW line and the dune foot. Eventually, the quantity supplied will be evenly distributed over the full height (and length) of the foreshore slope, following the equilibrium rules dictated by wave climate and grain size. This means that a large quantity of freshly supplied material will disappear under water soon after the nourishment operation. The public tends to call this erosion, though we must understand that it is initially no more than a re-distribution of the material within the natural cross section of the coast.

Placing of the sand on the dry beach is sometimes complicated because it is necessary to cross the breaker zone with the dredging equipment (see Figure 10-8). Since the breaker zone is an unfriendly place in which to work, this is a cost item cannot be neglected. This is the main reason to consider a fourth position of placement: on the foreshore. Again the idea is that the material is re-distributed from this position over the entire coastal profile. This works only if the material can be supplied in a region that forms part of the breaker zone. During calm wave conditions, material is dumped for this purpose by shallow draft dredges at the shortest possible distance from the beach (see Figure 10-8).


Figure 10-7 Beach suppletion using a pipeline that crosses the breaker zone


Figure 10-8 Foreshore suppletion

11. DREDGING

11.1 Introduction and definitions

Dredging is such a powerful tool of coastal engineering that it deserves special attention in lecture notes on coastal engineering. The capacity of modern dredges is so large that nowadays it is possible to cope with almost every situation. Transport rates of far over a million cubic metres per week can be achieved without too much of a problem. Still, it must be emphasised that it makes no sense to work against nature, although the dredging capacity may be sufficient to do this. It is much wiser to try to work with nature, and to try to adapt goals and working methods to accord with natural processes.

According to dictionaries, dredging is "deepening or widening a river, a harbour or a channel by removing sand, mud or silt." This definition focuses too much on one purpose: deepening of a location below the water level. It is equally important to include the supply of material to raise the level of a certain area.

Among Dutch civil engineers a frequently heard definition of dredging is "large scale transport of soil". This may be true in the Netherlands, but large-scale transport of soil can take place in at least two ways. One, by making use of rolling equipment (dry bulk transport) and another, making use of water as medium (wet bulk transport). It is the wet bulk transport that is termed "dredging". Using water as medium for the transport can again be done in two different ways: by using vessels to transport material, or by mixing the material with water and pumping it through a pipeline. The "wet" character of the operation can also be attributed to the location from where the material is obtained (borrowed): in dredging this material is generally from below the water surface. Conditions in the Netherlands make dredging to the most attractive form of bulk transport for soil. In other parts of the world, however, similar large capacities are realised with the aid of dry earth moving equipment.

Dredging can be considered from the point of view of the various types of equipment that are used in the dredging industry. However, it is also possible to describe the dredging cycle, from the digging of the earth to the deposition of the dredged material at the final location. When this approach is followed, it is practical to split the cycle into a number of processes that can be scientifically examined. When doing this, it must not be forgotten that in reality it is a continuous cycle, and that each process in this cycle is directly connected with the preceding and the following elements.

In this chapter, first, the dredging process will be analysed via a division in four sub-processes:

- Digging (breaking up the cohesion of the soil)
- Vertical transport
- Horizontal transport
- Deposition

For each process, various working principles will be identified, after which frequently-used dredging equipment will be classified according to the working principles of the various processes within the dredging cycle.

11.2 The world of dredging

According to the definitions, given above, whether or not they are very satisfactory, dredging plays a part in the construction or maintenance of much public infrastructure such as rivers, ports and harbours. This means that dredging operations are often controlled or carried out by or on behalf of public entities like Public Works Departments, and Port Authorities. These bodies either act as employer and contract the dredging works out to private parties (i.e. dredging contractors), or they decide to carry out the works themselves and use their own staff and dredging equipment.

Much has been said about advantages and disadvantages of in-house dredging. Here it is merely concluded, that many countries that used to favour in-house dredging (e.g. USA, France, Germany), have to a large extent privatised their operations, mainly for reasons of cost efficiency. Insofar as they have decided not to privatise to a 100%, they have at least administratively separated the responsibility for maintaining the infrastructure (client/employer function) from the actual execution of the works (contractor function).

Therefore, it seems appropriate to distinguish two parties in the dredging world, employer and contractor; whether they belong to the same organisation is not important in this respect.
The main tasks of the employer are:

1. to define the need for and the scope of works to be carried out
2. to collect relevant data that are essential for the planning of the works
3. to draw up specifications, float a tender and prepare contract documents; all in such a way that the best value for money is obtained
4. to check whether the works are being carried out (and completed) properly according to the specifications and the contract, and whether they contribute to the need as defined under a)
5. to check whether the cost claimed by the contractor is reasonable and see that the contractor is paid accordingly

As compared to the tasks of the employer, the tasks of the contractor are rather limited:

6. To acquire a sufficient number of jobs at such a price levels that the continued existence of the company is ensured.This looks rather simple, but it is more complicated, since the price level in the market is determined by competitive forces. If working methods or technology are changing and lead to cost savings amongst his competitors, any contractor will have to follow. Since dredging is capital-intensive and the lifetime of the plant is rather long, a contractor who wants to remain in business will have to:
7. Plan ahead and invest in research and training.

When one is considering the dredging world as a market place with sellers and buyers, it is as well to realise that it is a small market only. There are a limited number of sellers and a limited number of buyers, and these are more or less compelled to work with one another.

A third, and sometimes confusing, party is the manufacturer of dredging equipment. He will try to produce the best possible equipment to remain in business, knowing that if one contractor goes for improvement, the others have to follow. This forces him to be at the forefront of research and innovation. However, he lacks information about actual field experience, and will have to rely on good relations of trust with his main customers. He must ensure that he does not (purposefully or unwittingly) leak information from one contractor to another one.

Amazingly, consultants play only a minor role in the dredging world. Few of them have sufficient up-to-date expertise. This is mainly due to the fact that it is difficult and labour intensive to gain access to all information required.

11.3 Dredging process and dredging equipment

11.3.1 General

Dredging equipment is often classified according to its mobility *(stationary vs. non-stationary)* and according to one aspect of the dredging process *(i.e. suction dredge)*. This classification ends with names such as "stationary plain suction dredge". It is possible to discuss dredging technology on the basis of a description of the types of dredges.

A completely different type of classification starts from the phases of the dredging process indicated in Section 11.1. In this case, a distinction is made between different phases of the dredging process. This approach has the advantage that one recognises the potential of non-conventional solutions at an earlier stage. It also becomes easier to identify the limiting phase or phases that determine the overall output of the envisaged working method. In this approach types of dredges are less important than the entire dredging cycle.

In this chapter, we follow the second classification. This means that we merely discuss different solutions for the various phases of the dredging process. A more theoretical approach to each family of solutions will be presented in subsequent chapters. Finally, we will also describe the best-known types of dredges, with their specific advantages and disadvantages.

11.3.2 Breaking up the soil structure

Breaking up the structure of the soil involves parts (grains or lumps) being separated from their original texture and possibly preconditioned for the next phase of the dredging process. This breaking up is generally done in one of two ways, i.e. by mechanical force or by the force of flowing water. In this course, no attention is paid to chemical or physical means(blasting and vibration).

Flowing water
When considering the force of flowing water, it is evident that this force may be derived from suction or from pressure related flow conditions. The soil generally disintegrates into lumps or particles that are carried away by the current. This means that automatically water is mixed with the soil. The erosive action of suction is limited to an area very close to the suction mouth, and therefore the system is not used frequently anymore. The area influenced by a jet flowing from a nozzle is much larger. One is not always certain, however, of the direction the jet-induced flow will take when it meets obstacles like an undisturbed seabed. For this reason, plain suction dredges or dustpan dredges are often equipped with nozzles in the vicinity of the suction mouth. Both types of dredge disintegrate the soil structure when the suction mouth is pushed forward into the seabed. The plain suction dredge leaves a moon crater like landscape; the dustpan dredge leaves a flat bottom. Another example of the application of erosion by suction is the California type of draghead. In this case, free suction is avoided by creating a thin cleft between draghead and seabed. All these types of suction dredge have difficulty in working in cohesive soils.

A recent example of the use of erosive force by jetting is the new technique of water injection dredging. Here, the jets are used to create a dense soil water mixture that flows to deeper sections of the seabed under the influence of gravity. There are other applications for plain suction dredges and dustpan dredges. Recently, jet nozzles are also being more frequently fitted the dragheads of trailing suction dredgers.

Mechanical forces
When mechanical means of disintegrating the soil structure are used, steel blades (teeth) are common. They are pushed through the soil in a manner similar to the movement of a chisel in steel or wood. Subsequently, we will see that the theory behind the chiselling action has made a great impact on the insight of this method in dredging. Even someone who knows little or nothing about dredging will recognise one similarity, which is that: blunt chisels do not work well. When this technique is used, considerable forces are exerted on the chisel-type blade that has to force its way through the ground. These forces must be mobilised externally, either having an extreme weight of the dredge part or via a sophisticated anchoring system. Examples of dredges using this method are the grab dredge, the bucket ladder dredge, the backhoe, the cutter suction dredge, and even the trailing hopper dredge, at least when the draghead is fitted with cutting blades or teeth. In all cases it is useful to realise how the external forces are mobilised, and how they are guided towards the tooth. A striking difference between mechanical and all hydraulic means of disintegration is the fact that little or no water is added to the soil structure at this stage.

11.3.3 Vertical transport

In most cases, the soil is brought to the surface after disintegration. Again, hydraulic or mechanical means can be employed. If mechanical means are used, no water has to be added to let the mixture flow. Examples are the grab dredge, the bucket ladder dredge and the backhoe. The process is relatively slow, but it does not require much energy, as no added water has to be raised to the surface. This is the main advantage of the method, which is still used in spite of the low production rates. For the purpose of this course, the process is not very interesting. However, hydraulic vertical transport is well worth scientific analysis. With the aid of the vacuum created by a pump (usually a rotary centrifugal pump), slurry is moved up through a pipe. This means that water has to be added to the soil to allow it to flow. Adding too much water leads to inefficiency, and strangely enough, the same applies if insufficient water is added. The method is used in all suction type dredges: plain suction, cutter suction, and trailing suction dredges.

11.3.4 Horizontal transport

Horizontal transport of the dredged material can be accomplished by barge, by pipeline, by truck, or less commonly by conveyor belt. Transport barges can be used in combination with the dredging equipment, or be used as separate units. It is evident that disintegration and vertical transport methods that do not add water to the soil have a great advantage. When water is added, transport costs can be reduced if water and soil can be separated during the loading of the barge, This may require a considerable effort, which will be discussed in more detail later. Barge transport is cheap (low cost per ton/kilometre), provided sailing conditions are reasonable. This refers to available water depth, currents, and waves. If dredging equipment and barges are separate, the loading process itself is also sensitive to delays.
Pipeline transport is commonly used with dredges that use hydraulic means for disintegration and vertical transport, though there are examples of material being brought up by a grab and then pumped to its destination. The advantage of pipelines is that they can cross both, water and land. No rehandling of material is required when moving from water to land or the reverse. Pipelines crossing water may be floating (either on pontoons or self-floating) or submerged. Floating pipelines must be flexible (rubber hoses or ball joints), but they remain sensitive to the action of waves and currents, and they hamper navigation. Submerged pipelines are welded together from steel pipes and sunk in place in lengths of up to several kilometres. This requires a considerable investment and generally, pipeline transport is more expensive than barge transport. Rehandling becomes cost effective when more then 4 to 6 km can be covered by barge. The theoretical aspects of pipeline transport will be discussed separately.
Truck transport is relatively expensive and only used for small quantities or over short distance, unless there are no other options. The use of trucks is a logistic rather than a technical issue.

Conveyor belts are not commonly employed in dredging projects either, though Japanese contractors have successfully used them on reclamation jobs in Singapore and Japan.

11.3.5 Deposition

The deposition of dredged material is carried out for different purposes and in different environments. The most important question is whether the material will be used in productively or merely be discarded. In the latter case, one is only concerned that the material does not finish-up in undesirable places, or that the disposal has undesirable side effects on the environment. When the material is to be retained, it makes a difference whether the location is on land or in water. The methods depend largely on the limitations imposed by the means of horizontal transport. Provided the depth permits barges are often unloaded by dumping through bottom doors,. In other cases, the material is pumped, which again means that during deposition, water and soil must be separated. This may be done in an unconfined or in a diked disposal area. When using diked disposal areas, it is possible to achieve considerable height of reclamation with relatively steep slopes. However at all times the rules of soil mechanical stability must be observed, taking into account the special properties of the high-density fluid that we are working with. The phenomena occurring during disposal are similar to those that can be observed when loading barges.

11.3.6 Back to one process

In spite of the advantage of splitting the dredging process into phases to enhance understanding, one must keep in mind that in reality the various phases are part of a single operation. This implies that all material that is disintegrated will have to be lifted vertically, transported horizontally and eventually deposited. Since it is a more or less closed circuit, in all phases the capacity should be approximately equal. There is no sense in increasing the capacity of one phase if the others remain unchanged. This is explained in the following example.

Example:

When a trailing suction dredge has two dragheads each 2.5 m wide, one can select the cutting depth of the draghead and the sailing speed during dredging.

If a cutting depth of 0.1 m and a sailing speed (during dredging) of 2 knots (= 1 m/s) are chosen the volume of material loosened from the seabed is:

$2 * 2.5\ (m) * 0.1\ (m) *1\ (m/s) = 0.5\ m^3/s.$

If the two pumps have a capacity to pump a mixture of 25% concentration at a speed of 4 m/s through two pipes of 0.75 m, the pumping capacity is:

$2 * 1/4\pi\ D^2 * 4 * 0.25 = 0.88\ m^3/s$

The pumping phase has a higher capacity than the digging phase. The production can be increased by applying either a higher sailing speed or a greater cutting depth.

12. USE OF THEORY IN DREDGING

In dredging technology, some special aspects of soil mechanics and fluid mechanics require special attention. With regard to soil mechanics, it is no longer stability that is important, but rather the loss of stability. With regard to fluid mechanics, flow in closed circuits and flow with extremely high sediment concentration are subjects just outside the traditional field that are nevertheless very important.

12.1 Soil mechanics

12.1.1 Classification of soils

In the dredging world, the term 'classification of soils' often refers to a description of soils to be used in official documents. The PIANC Report no. 47 (Anonymous, 1984) is a frequently used international standard for this purpose. This report recommends methods for identification and subsequent (laboratory) investigations to assess the soil properties relevant for dredging. Loose granular material is usually classified according to grain size:

Name	Grain size (mm)
Boulders	> 200
Cobbles	60 - 200
Gravel	2 - 60
Sand	0.06 - 2
Silt	0.002 - 0.06
Clay	< 0.002

Table 12-1 Classification loose granular material

Apart from the grain size, other data are important including grain density (ρ) pore volume, and parameters describing the plastic behaviour of silt and clay. Bedded rocks are treated in a different way and for their classification one should refer to the original publication.
The grain size is generally determined by sieving, but for finer particles it is not uncommon to determine the grain size by measuring the fall velocity of the particles in water. This method is based on the well-known Stokes equation, $w \approx d^2$, valid for small Reynolds numbers, i.e. for fine grains only.
This equation can also be represented in graphical form as demonstrated in Figure 12-1, in which γ_g (the grain density) has been taken as 2650 kg/m^3 and γ_w (the density of water as 1000 kg/m^3).

12.1.2 Porosity and bulk density

Granular soil consists of a mixture of grains, air and water. An important parameter is the porosity (n), indicating the volume of the pores (V_p) divided by the total volume (V_s). The estimated porosity usually varies between 35 and 45 % with 40%. In some countries the ratio $e = V_p / V_g$ is used, which relates easily to the void ratio: $e = n / (1-n)$.
Taking the density of the grains as 2650 kg/m^3, and assuming n = 40%, it is easy to see that the bulk density of dry soil is about 1600 kg/m^3. If the voids are saturated with water this increases to 2000 kg/m^3.

12.1.3 Permeability

The voids in a granular soil are linked with each other. This means that under the influence of an external force, water can flow through the voids. In **fine-grained** material, this is a laminar flow, because the friction of the flowing water is mainly determined by viscosity. Therefore, the Darcy equation can be used:

$$u_f = k \cdot i \tag{12.1}$$

in which
u_f = filter velocity (m/s)
i = gradient of the static head in the direction of flow (-)
k = permeability coefficient (m/s)

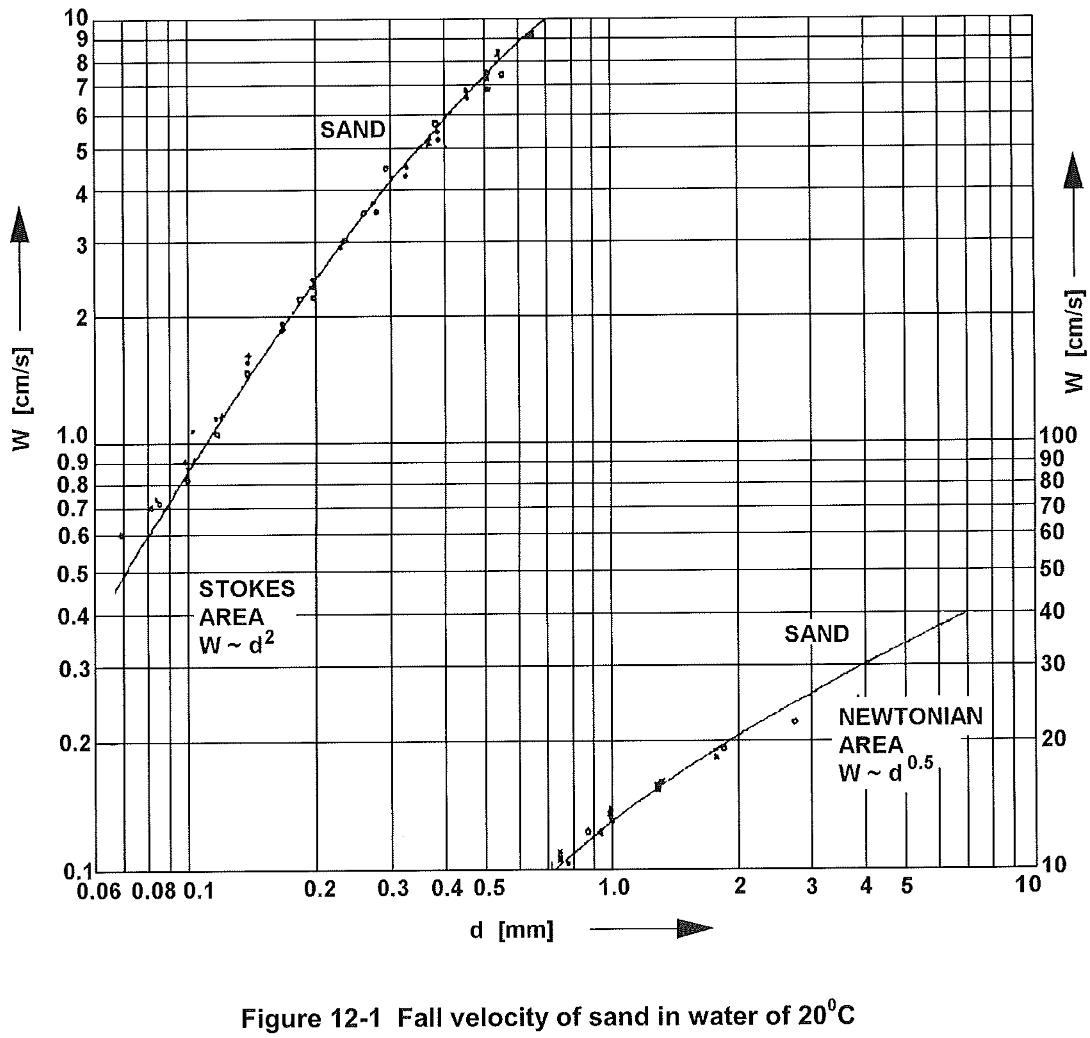


Figure 12-1 Fall velocity of sand in water of 20°C
Note: d in mm and w in cm/s!

The filter velocity is a fictitious velocity, defined as the flow rate Q (m^3/s) divided by the total area A (m^2). Because grains occupy a part of the area, the actual velocity u_p in the pores will be considerably higher than the filter velocity. ($u_f = n.u_p$) Permeability plays an important role in dredging, and therefore it would be ideal if permeability could were measured more frequently during surveys. However it is possible to determine the permeability on the basis of the grain size distribution with the aid of the Kozeny-Carman relation:

$$k = 1/5 \frac{g}{\nu} \frac{n^3}{(1-n)^3} \frac{d_{50}^2}{36} exp^2 \left\{ -1/4 \, ln^2 \left(\frac{d_{85}}{d_{15}} \right) \right\} \tag{12.2}$$

in which:

d_x = characteristic values from grain size distribution (m)
ν = kinematic viscosity of water (m^2/s)
n = porosity (-)
g = acceleration of gravity (m/s^2)

As the flowing fluid experiences a friction when passing through the voids, one must expect (action = reaction) that conversely the same force is exerted by the fluid onto the medium. This flow force is equal to:

$$p_f = \rho_w \cdot g \cdot i = \rho_w \cdot g \cdot \frac{\partial h}{\partial x} \tag{12.3}$$

The dimension of p_f is force per unit volume, which plays an important role when we have to consider the equilibrium of slopes with groundwater flowing out.

12.1.4 Stresses

Terzaghi has treated stresses in soil extensively. He defines a vertical stress level in the soil that is equal to the total weight of the overlying soil (i.e. grains + void water) [N] divided by the area [m^2] that carries the load. This stress is called the total stress σ_{tot}, or simply σ. However, part of the vertical force must be transmitted by the hydrostatic (water) pressure, part by forces concentrated in the contact points between the grains. If the water pressure is p, this means that the fictitious effective stress or "grain pressure" $\sigma' = \sigma - p$, or:

$$\sigma = \sigma' + p \tag{12.4}$$

This formula is only valid for normal stresses. Friction or shear cannot be transferred by water, so it must be transferred in the contact point between the grains, where the normal stress level is σ'. The maximum shear that can be transferred is proportional to the friction coefficient (tan ϕ) multiplied by the effective stress:

$$\tau_{max} = \sigma' \tan \phi \tag{12.5}$$

This expression has large consequences: if the water pressure increases or decreases in otherwise identical conditions, the effective stress will have the reverse action in each case, as will the shear strength.

$$\tau_{max} = \sigma' \tan \phi + c \tag{12.6}$$

The above considerations apply for granular material where friction forces dominate. In finer material like silt, and in particular clay, other forces (such as electrostatic loads) dominate the inter-grain behaviour. This leads to an extreme situation, where the shear strength no longer depends on the normal stress, but exhibits a constant value: the cohesion (c). Equation (12.6) then becomes:

$$\tau_{max} = c \tag{12.7}$$

In most examples in this course, we simplify the behaviour of material with grain sizes larger than 0.06 mm to a condition with $c = 0$ and $\phi = 30^0$ (non cohesive material), and those with grain size < 0.002 mm to c = constant and $\phi = 0$.

12.1.5 Deformations

Soils do not behave exactly the same way as materials like steel and wood when deformation under the influence of loads occurs. While steel and wood show a largely elastic behaviour, soils behave rather differently. This is because changing the packing of grains causes deformations. The denser the packing, the stiffer the soil. Increasing normal stress will cause denser packing and thus of stiffening behaviour. Another striking difference is that a release of stress does not lead to looser packing. When the soil is recharged to its original stress level, it will therefore show an extremely stiff behaviour and if it is loaded beyond the original stress level, it yields easily. We call this over-consolidation. In nature, this effect may be caused by different configuration in previous geological periods. In practice, we apply this property by surcharging the soil temporarily to reduce settlements in later stages.

When soil is loaded, and thus is compacted, the void ratio n will decrease. In saturated soil, this means that ground water has to move out. This will take time, (certainly in fine-grained material), because of the low permeability. Consequently, there will be a considerable period during which the water pressure will be higher than original pressure and possibly also considerably higher than the usual hydrostatic value. This means that the effective stress and consequently the shear strength are reduced (equations (12.4) and (12.5)). When the water pressure becomes higher than the total stress, the effective stress becomes zero, and the soil loses all ability to withstanding shear. This is called liquefaction, or the formation of quicksand.

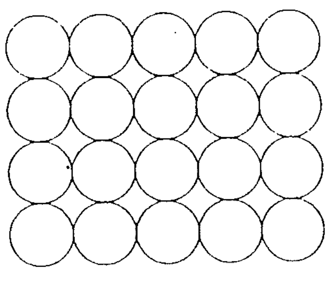


Figure 12-2 Loosely packed material

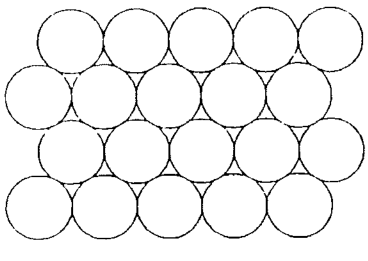


Figure 12-3 Densely packed material (dilatant)

When deformation takes place under the influence of shear rather than under the influence of normal stresses, the situation may be different. If the grains are loosely packed (Figure 12-2), it will be easy to form a plane where particles can slide along each other. If the grains are densely packed (Figure 12-3), grains have to move away from each other before sliding can take place. This means that the void ratio n is increased. In saturated soil this means that water has to flow into the slide plane. If this is hampered by low permeability, considerably lower pressure may occur locally than the original values or the hydrostatic value.

This leads to an **increase** in the effective stress and thus to a higher shear strength. This effect is called **dilatancy**, and it plays a dominant role in various parts of the dredging process. There is a limitation, however on the under-pressure that can be generated: if pressures fall below the absolute vacuum, water will change into water vapour, and expansion can take place easily. The "pre-stressing" water pressure is thus limited to 10 mwc (100 kPa) below the atmospheric pressure.

12.1.6 Stability of slopes

When considering the equilibrium of a slope the theories of Fellenius and Bishop are commonly used. Both methods divide the ground mass into vertical slices. The difference between the methods of Fellenius and Bishop is the complete negation of forces between the slices by Fellenius. Loss of stability can occur only when a circular body moves along a straight or a circular slide plane. In the case of a slope, the circular slide plane is the most common. The stability is calculated by taking the turning moment around the centre *M* of a slip circle with radius *R* (Figure 12-4). Destabilising forces are the weights of the soil slices on the right side of *M*, while stabilising forces are the weights on the left of *M* plus the friction in the slide plane. The ratio between stabilising and destabilising forces can be calculated for each location of *M* and value of *R*. Values above 1 indicate stability. For permanent slopes, a value of 1.3 to 1.5 is recommended as a minimum safety, for temporary slopes, values as low as 1.1 may be accepted.
In a homogenous non-cohesive material, many methods of calculation indicate that a slope equal to the friction angle (or angle of repose) is on the boundary between stable and non-stable slope. Examples of loss of stability of slopes exposed to real life dredging hazards are given in Schiereck (1993).

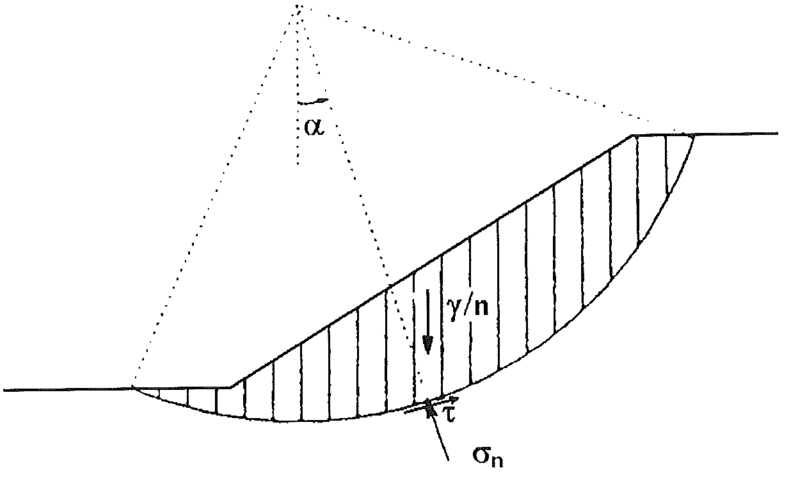


Figure 12-4 Slide plane calculation

12.2 Hydraulics

12.2.1 General

Fluid Mechanics may be a more appropriate title of this chapter, but term *hydraulics* is used here, because fluid mechanics is not treated here on a scientific basis. This is merely an attempt to refresh the memory of the reader in a very practical and applied way. Subsequently, attention will be paid to open channel flow, flow in closed conduits, sediment transport and pumps.

12.2.2 Sediment transport in open channels

The friction that water encounters when it flows along a wall must be equal to the shear force that is exerted by the flowing water on that wall. Consequently, the value of the bottom shear stress τ_o, or the shear velocity U_*, must play a role in considerations of sediment transport. Shields found that there is a threshold shear stress, below which virtually no sediment transport takes place. The critical value of the Shields parameter $\psi = U_*^2 / \Delta g d$ appears to be a function of the Reynolds number related to the grain size $U_* D / \nu$ (See Figure 12-5). Note: Δ is the relative density of the bottom material: $(\rho_{grain} - \rho_{water}) / \rho_{water}$.

When the critical value of the Shields parameter is exceeded, the current will move grains. The rate of transport has been subject of numerous investigations, yielding widely varying results. At least part of the scatter is caused by the fact that the transport takes place in different manners, i.e. partly along the bottom, partly in suspension. It is likely that the mode of transport depends on the ratio between U_* and w, the fall velocity of the particles. Well-known expressions are those of Meyer-Peter and Müller (for bedload only) and Engelund and Hansen (for bed load plus suspended load). The formulae may be generalised into a form:

$$S_b = B\, m\, U^n \qquad (12.8)$$

in which:
B = width of the channel [m]
m = coefficient
n = coefficient (as high as 3 to 5)
U = average velocity [m/s]
S_b = bulk sand transport [m^3/s]

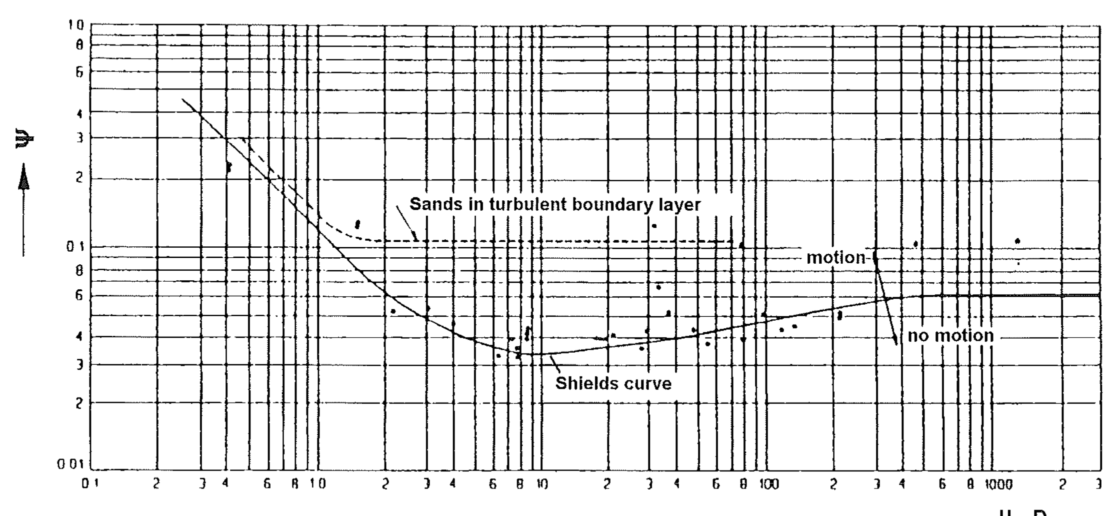


Figure 12-5 Shields' curve

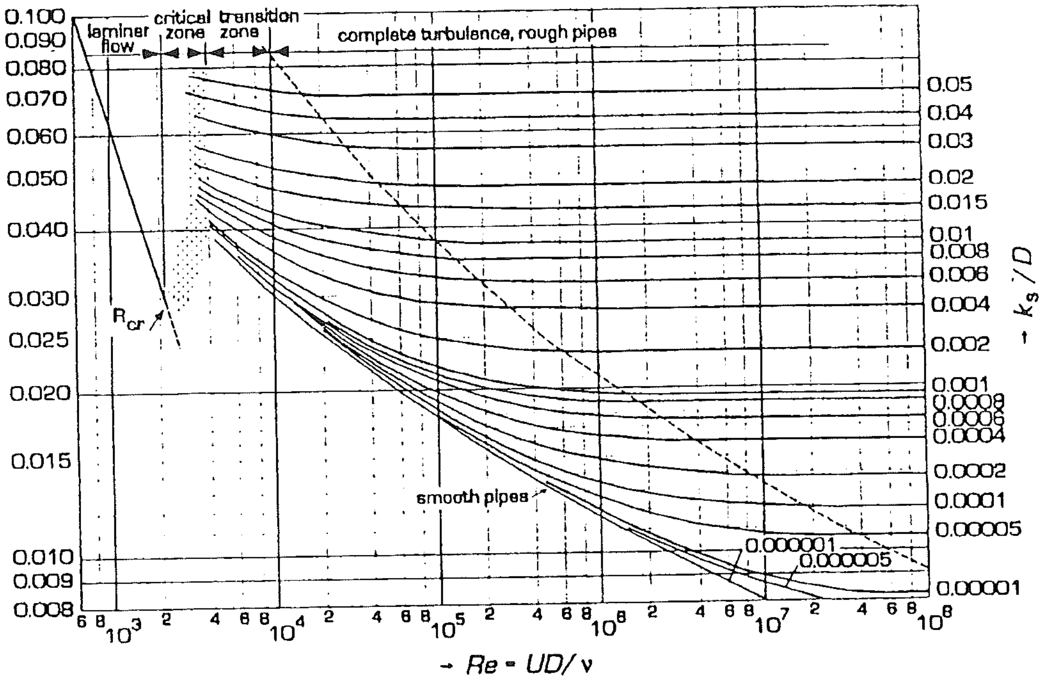


Figure 12-6 Moody Diagram

Bulk transport must be discerned from grain volume: $S_b = S\ /\ 1\text{-}n$ if n is the porosity.

It must be kept in mind that these theoretical values apply for the friction of flowing water over an almost horizontal bottom. When water flows over or along a steep slope that is already close to the limit of stability (relating to soil mechanical transport aspects) and threshold conditions will differ considerably.

12.2.3 Flow in closed conduits

The flow in closed circuits can be treated in a way similar to the flow in open channels. For wall friction, a relation can be developed that is similar to the Chézy formula:

$$\Delta H_v = \lambda \frac{L}{D}\frac{U^2}{2g} \tag{12.9}$$

in which:

ΔH_v = drop of the energy level over a distance L [m]
λ = friction coefficient [-]
L = pipeline length under consideration [m]
D = pipeline diameter [m]
U = average velocity [m/s]

The friction coefficient λ can be taken from a publication by Moody (Figure 12-6). Comparing the work of Moody and Chézy, it is evident that $\lambda = 8g/C^2$. When using the Moody diagram for steel pipelines, the value of k_s can be taken as 1 mm or similar. However, losses are not due to friction alone. As in open channels, extra losses may occur due to local disturbances. In the case of pipelines, this refers amongst others to inflow and outflow losses, bend losses, losses due to valves and junctions. Many of these anomalies have been tabulated, and one may attach a typical extra resistance to each discontinuity, i. The extra resistance is expressed as:

$$\Delta H_i = \xi_i \frac{U^2}{2g} \tag{12.10}$$

The total head loss over the pipeline is thus:

$$\Delta H_{total} = \sum_{i=1}^{n} (\xi_i \frac{U^2}{2g}) + \lambda \frac{L}{D} \frac{U^2}{2g} + \Delta H_{static} \tag{12.11}$$

In this way, it is possible to construct a *Q-H* curve for any closed conduit (Figure 12-7). By combining the *Q-H* curve of the pipeline with the *Q-H* curve of the pump, it is possible to find the actual working point of the pump. For details about the working of a centrifugal pump, the reader is referred to Annex 6.

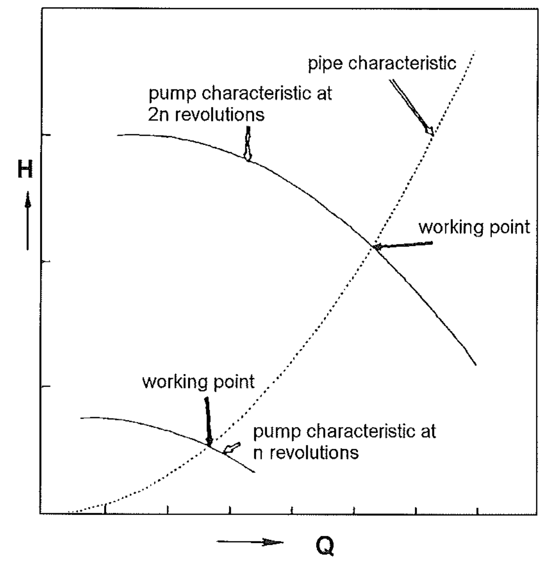


Figure 12-7 *Q-H* curve for pump-pipeline interaction
Note: ***Dashed line is Q-H curve for pipeline***
Drawn curves are *Q-H* curves for pump action at two different speeds

12.2.4 Sediment transport in closed conduits

When sediment is added to water flowing in a pipeline, it is important to define parameters in addition to those given above. The diameter of the sand grains is taken as d (m), their density as ρ_g, their fall velocity as w. The velocity of the mixture is defined as U_m, and the pipeline diameter as D. The transport concentration C_T is defined as Q_{sand} / Q_{total}, in which Q_{sand} accounts for the grain volume only. Sometimes the volumetric concentration C_V is used. Since there is often some slip between the lowing water and the slower moving sand particles C_V must be larger than C_T. The ratio between the two is called the transport factor $\alpha_T = C_T / C_V \leq 1$.

In practice, the dredging industry prefers to use the bulk concentration $C_B = C_T / 1 - n$. The use of the bulk concentration leads immediately to actual volumes in the dredging or reclamation area. Taking ρ_g as equal to 2650 kg/m^3, and n as 40%, the density of the mixture is a direct indication of the bulk concentration.

Example:

If the bulk concentration in a mixture is 20%, this means 80% of every cubic meter of mixture is water, and 20% is soil(bulk). The bulk soil also consists partly of water (40%) and partly of grains (60%). The weight of one cubic meter of mixture is thus:

water direct:	0.8 m^3 @ 1000 kg/m^3	800 kg
water in voids:	0.08 m^3 @ 1000 kg/m^3	80 kg
grains:	0.12 m^3 @ 2650 kg/m^3	318 kg
Total		1.198 kg
or rounded off		1.200 kg,

which is 20% extra (compared to the density of the water)

Now, it is possible to discern various materials flowing through the pipe. When a high concentration of clay or silt is pumped, this will influence the viscosity of the mixture so much, that the normal theories for turbulent flow do not apply.

When fine sand is pumped, the fall velocity will be very low compared to the turbulent velocity fluctuations in the pipeline. The latter are of the order of 5% of the mean flow velocity. For example, if we assume that in order to create a more or less homogenous mixture these turbulent fluctuations must be a factor 10 greater than the fall velocity and if we assume a reasonable velocity in the pipeline of 4 m/s, the fall velocity of the particles must then be lower than 0.02 m/s, or the diameter < 0.15 mm. In other words, fine sand will form a more or less homogenous mixture provided the diameter is between 0.06 and 0.15 mm.

If we pump coarser material, part of the material will remain on the bottom. No material at all will come into suspension if the turbulent velocity fluctuations are in the same order as the fall velocity. All material will thus be transported along the bottom of the pipeline if the fall velocity is in the order of 0.2 m/s, i.e. if the grain size is more than 2 mm. Thus, we may conclude that grain sizes between 0.15 and 2 mm form a transition zone, and that material coarser than 2 mm will slide over the bottom of the pipe. Consequently, one may expect that for sand grains under 0.15 mm, the transport coefficient will be 1, whereas for material coarser than 2 mm the transport coefficient may even drop to values below 0.5. Exact data are not readily available in the published literature.

Apart from the question of exactly how the grains are moving, it is important to ensure that they do not remain behind in the pipeline and form a growing obstruction to the flow. The limit velocity for this phenomenon is called the critical velocity. Many investigators have made observations in model test rigs, but their results vary widely. This is partly because the definition of critical velocity is not very accurate. Widely accepted data were been published by Führböter (1961) and by Durand (1952) as far back as 1952 and 1961. From these publications, one may conclude that the parameter $\sqrt{\Delta gD}$ plays a dominant role. This makes it obvious that the critical velocity increases with $d^{0.5}$. This conclusion is valid for coarse material, but for finer material, a power of 0.2 is also mentioned in the literature. Apart from that, the grain size d makes little difference when d > 1mm. (*More recent investigations raise some doubt about these processes, certainly for material with grain sizes between 1 and 3 mm*). The concentration is important for values of C_T< 0.15. In practice, we try to keep concentrations higher than this value. A very important factor is the grain size distribution. The addition of finer material will certainly increase the critical velocity. Practical values for a pipeline of D = 0.65 m are:

d (mm)	U_{crit} (m/s)
0.1	3.2
0.15	3.6
0.2	4.3
0.3	5.0
0.4	5.5

Table 12-1 Critical velocities for a pipeline D = 0.65 m

With respect to the pipeline resistance, it makes a big difference which mode of transport occurs. For *fine sand,* i.e. d between 0.6 and 0.15 mm, the resistance is proportional to the density of the fluid. The result of equation (12.11) must be corrected by a factor ρ_m / ρ_w. In the *transition zone,* the Führböter formula is often preferred. This relates the gradient when pumping mixture (i_m) with the one found for water (i_w), in the following way:

$$\Delta H_m = \Delta H_w + \frac{S_{kt}}{U_m} C_T L \tag{12.12}$$

in which S_{kt} is a coefficient that can be calculated with the following expression:

$$S_{kt} = 2.59 \cdot 10^3 d_m - 0.37 \tag{12.13}$$

and

$$d_m = \frac{d_{10} + d_{20} + ...+ d_{90}}{9} \tag{12.14}$$

For *coarse sand* (d > 2 mm), the Durand formula seems to fit better with field observations. The general form of this formula is:

$$i_m = i_w + 176\, C_T \cdot \frac{\lambda_w}{2} \cdot \left(\frac{g D}{U_m^2}\right)^{0.5} \cdot \left(\frac{w}{\sqrt{g d}}\right)^{1.5} \tag{12.15}$$

After some modifications, the Durand formula can be converted into:

$$i_m = i_w \left\{ 1 + 294 \left(\frac{g D}{U_m^2} \right)^{1.5} \right\} \tag{12.16}$$

With the above theories, pipeline resistance curves can be drawn for various velocities and concentrations.

In the mean time, more up to date literature is becoming available, in particular the work of Wilson. This leads to slightly lower resistance values for sand in the range of 1 to 2 mm diameter. Use of the Wilson theory requires computer calculations, which makes the method less useful for the purposes of instruction.

13. DREDGING CYCLE

13.1 General

In this chapter the phases of the dredging process as introduced in section 11.3 are elaborated quantitatively.

13.2 Disintegration

13.2.1 Suction

When plain suction is applied as means for disintegration, soil is removed from its place when the suction pipe is extended into the ground. Material slides down the slopes around the suction pipe until stable slopes are achieved. When production has to continue thereafter, one must push the pipe ahead, and cause a continuing instability at the forward end of the trench created by the suction pipe. In many cases, sliding causes dilatancy and thus under pressures in the soil, certainly when the original packing is dense and the permeability low. This means that the sand in front of the pipe is pre-stressed. It becomes as hard as concrete. Only slowly, do large lumps fall down after sufficient water has passed into the voids. One can imagine that the maximum forward speed of the suction mouth is therefore restricted by the permeability of the soil. The production of the trench is equal to its cross sectional area multiplied by the forward speed of the suction mouth (compare example 1). The production can thus be limited by the limitation of the forward speed, which is a function of permeability. Another method of increasing trench production is to increase the cross sectional area of the trench, in other words: push the suction pipe deeper into the ground. Therefore, it is common to see plain suction dredges dredging to depths of 60 or 70 m.

13.2.2 Jets

The flow field created by jets is extensively described in the literature. For a summary, the reader can refer to Schiereck (1993). Unfortunately, the effect of this flow pattern on the erosion of granular soil is not described in the literature within the public domain. From the presence of jets on the suction pipes of all major plain suction dredges, one may conclude that this is effective. A first impression of required jet power might be obtained by comparing the installed jet power with the power of the suction pump of such dredges. This leads to the conclusion that the jet power should be around half the power of the suction pump. Where theoretical knowledge is not available, one may rely on the experience of others.

13.2.3 Blades

When cutting soil by mechanical means, a slide plane forms in front of the cutting blade. In principle, it is not very difficult to calculate the forces along this slide plane by using the theories of traditional soil mechanics. However, there are complications when the soil is saturated with water. Again, dilatancy may occur in the slide plane, and as the speed of the blade is rather high (trailing suction hopper dredges: 1 to 2 m/s; cutter suction dredges: up to 3 m/s), water has very little time to flow to the dilatant zone, even though the cutting depth may be small. Theoretical considerations have demonstrated that in impermeable sand the under pressure may reach the cavitation point (-10 mwc). These calculations have been confirmed by measurements in the laboratory.

Extensive research has been done, but only little has been published (Meijer, 1976 and Miedema, 1987). It has indeed been demonstrated that it is possible to model dilatancy and the resulting water under pressures and to calculate the horizontal and vertical forces on the blade (Figure 13-1). Results of this kind of research can be applied to all kinds of chiselling tools like dragheads, cutterheads, buckets, and bucket wheels. It is important to note that in the case of cavitation the water depth in the dredging area largely determines the required forces and energy.

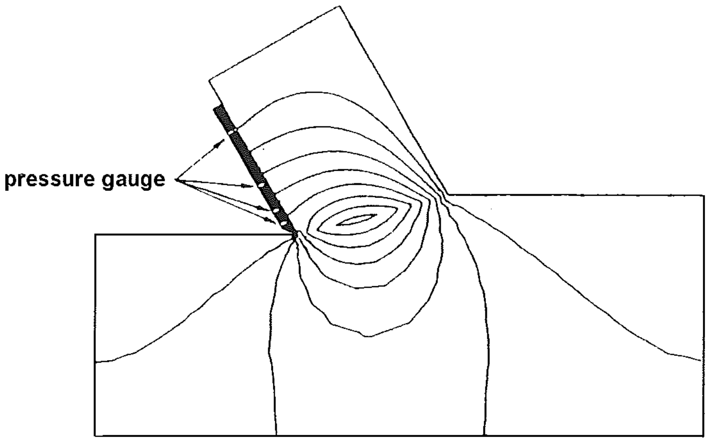


Figure 13-1 Cutting blade with contours of (neg.) water pressure

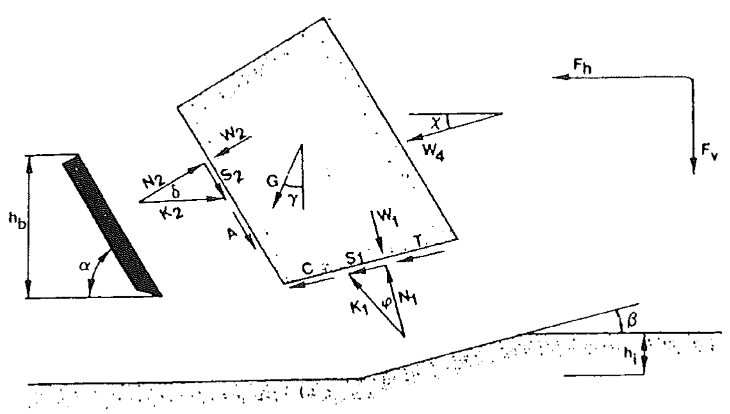


Figure 13-2 Forces on cutting blade

13.3 Vertical transport

13.3.1 Mechanical transport

Cranes provide the mechanical means for vertical transport. Lifting is mainly done by using winching wires or by using hydraulic cylinders to drive an arm. Another typical example of mechanical vertical transport is the chain of a bucket ladder dredge.

13.3.2 Hydraulic transport

More important is the use of hydraulic means for the vertical transport of dredged material. Here we consider a pump with its centre at a level z_p below the water surface. It is connected with a suction pipe that ends at a depth z_s below the water level. At its suction side, the pump can generate a suction pressure p^* (in mwc). (Figure 13-3)

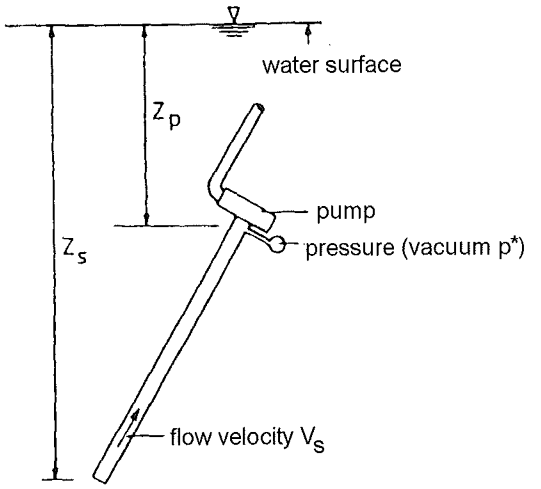


Figure 13-3 Definition sketch of terms used in the suction equation

When we consider the equilibrium of the column of fluid in the suction pipe, we can recognise two upward forces:

- the pressure at the suction mouth: $Z_s.\rho_w$ and
- the suction by the pump: $p^*.\rho_w$

We also can distinguish downward forces:

- the weight of the fluid in the suction pipe: $(Z_s - Z_p)\rho_m$
- friction and other hydraulic losses: $(fU^2 / 2g)\rho_m$

Since there must be equilibrium, the best known equation in dredging reads:

$$(p^* + Z_s) \cdot \rho_w = \left(Z_s - Z_p + f\frac{U^2}{2g} \right) \rho_m \qquad (13.1)$$

In practice, p^* is limited to a value well below the absolute vacuum, say 0.55 m Hg, or 7.5 mwc. Equation (13.1) cannot always be solved directly so this has to be done by iteration. Nowadays, with the aid of spreadsheets, this is no problem.

An example of the kind of problems that can be solved is given in the following example.

Example:

A dredge with a suction pipe of 0.7 m diameter and its pump at the water level is dredging at a depth of 20 m. The maximum vacuum of the pump is 7.5 mwc, the density of the water is 1000 kg/m^3. What is the optimum production?
Using equation (13.1) and substituting the data, we find:

$(20 + 7.5).1000 = 20.\ \rho_m + 3.\ U^2/2g\ .\ \rho_m$ (if we assume f=3)
or
$27{,}500 = (20 + 0.15\ U^2).\rho_m$

This equation has two unknown variables. Therefore, we must assume a value for U. Take subsequently 2.5, 3, 3.5, 4, and 4.5 m/s.
This yields mixture densities of respectively 1320, 1290, 1260, 1230, and 1190 kg/m^3, and thus **bulk** concentrations of 32, 29, 26, 23, and 19%.
To find the bulk production in sand, we must multiply the discharge Q by the bulk concentration, i.e.
$1/4\ \pi\ D^2 * U * C_B * 3600$ (seconds per hour).

This leads to the following tabulated result:

U (m/s)	C_B (%)	Production (m^3/hr)
2.5	32	1090
3.0	29	1180
3.5	26	1250
4.0	23	1260
4.5	19	1170

The optimum suction production is achieved for a suction velocity of about 4 m/s.

Considering the formula (13.1) more carefully, one will notice that increasing the suction depth will soon lead to unacceptably productions. This can only be solved when the depth of the pump below water level (Z_p) is increased. This can clearly be noticed when one is on board a trailing suction hopper dredge: the suction process runs better after the vessel's draft has increased owing to loading.

13.4 Horizontal transport

13.4.1 Pipeline

In section 12.2.4, close attention was paid to the theory of pipeline transport. An aspect that has not been mentioned yet, is the influence of the pump behaviour. This influence was thoroughly investigated by Stepanoff and his results are published in his classic book on pumps. Depending on the actual design of the pump, it is possible to calculate its Q-H curves for various speeds and mixture concentrations.
Now, all ingredients are available to draw pipeline curves for various flow rates and various mixture densities, and do the same for the pump curves, corrected for drive power and mixture

influence. The intersection of pump curve and pipeline curve gives the working point of the system (see Figure12-7).

Again, optimisation involves many iterative calculations. Dredging companies have made these calculations for all their dredges, and for a convenient set of pumping distances and grain sizes. Frequently they have adapted the theoretical values given in this course by actual calibrated measurements carried out while working on during their own projects. Clearly this information is a closely-guarded secret.

13.4.2 Barge

A very controversial phenomenon is the process that occurs during the filling of barges by hydraulic dredges. Filling the barge with the mixture as it is pumped leads to small loads, 20 to 30% of the maximum possible load. This means that pumping is continued after the barge is already full, and that the barge is used as a sand trap or settling chamber. The intention is that the water should flow overboard, and that the sand should remain in the hold. Thus, we can compare the working of the barge with the traditional sand trap (Figure 13-4).
When a sand water mixture is flowing at a rate Q into a rectangular settling basin with a width *B*, a depth *h*, and a length *L*, the flow velocity *U* in the basin is *Q/B.h*. When the fall velocity of the sand is *w*, a sand particle that was originally at the surface will settle within the basin if $L \geq U/w$. h. If we substitute U=Q/Bh, we find $L \geq Q/Bw$, or $w < Q/BL$. Apparently, the value of Q/BL, which is a fictitious vertical velocity in the basin determines the quality of the sand trap. One would therefore expect that for a given barge, reduction of the discharge would improve the sand retaining properties. It is also clear that an increase in grain size has a large influence on the efficiency of the filling process, certainly in the lower ranges of grain size, where the fall velocity increases with the square of the grain size (compare Figure 12-1).

When filling a barge the dilemma is clear, if one pumps the mixture at high capacity, the barge will be filled rather quickly, but the sedimentation during overflow will be less efficient. What is the optimum discharge? The problem is more complicated than it initially appears to be, as it is also known that higher concentrations lead to lower fall velocity. The next point is the question how long one should continue loading. Gradually more and more sand will flow overboard, along with the pumped water.

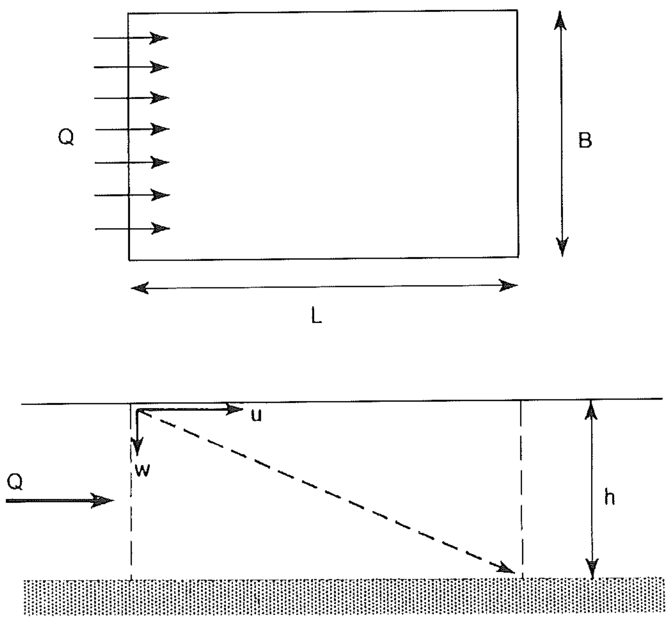


Figure 13-4 Settling particles in sand trap

Is it worthwhile to continue pumping to achieve a small increase in payload? This latter question cannot be considered on its own. When the barge has to sail a short distance it is easier to accept a poor load than when it has to make a long trip. Generally, the cycle time analysis will be used for this problem. Note that this method is often applied in the wrong way, taking the empty vessel as a basis for optimisation. Instead, one should base the optimisation on the vessel when fully loaded with water only.

The barge loading process is again at the centre of attention, not only during operational activities, but also for designing of new equipment.

Barge loading may bring further advantages and disadvantages. An advantage is the improved quality of the barge-loaded sand, when used as fill material. If the virgin material contained too much unsuitable material, this will be lost during the barge filling operation because the fines will easily flow overboard. This indicates the disadvantage as well. In sensitive areas, where increased turbidity is not acceptable (coral reefs, tourist beaches), one must be careful when considering using the method. Moreover, there is also a risk of unwanted sedimentation.
In industrialised areas, one must be extra careful. When heavy metals or hydrocarbons are discharged into the water, these harmful substances become attached to the finer particles. The dredging and transport of such contaminated material may have negative environmental impact. In many ports in the USA and Western Europe, maintenance-dredging operations (if permitted at all) are hampered by the cost of remedial measures.

13.5 Disposal

The disposal of dredged material can be performed in many ways. Basically, it is possible to distinguish methods by which the material is discharged directly from barges, and methods by which the material is discharged more gradually, by pipeline, or by grab (crane).

When discharging directly from barges, it is evident that the depth of the water at the disposal location must be sufficient for safe unloading. In many cases the required depth is greater than the actual draft of the barge, since bottom doors or valves protrude from the keel before the load is actually discharged. During the unloading process, the draft will rapidly decrease, so that there is little risk of the barge touching the bottom after the load has left the vessel. When the intention is to create an under water berm of a particular shape, one must be alert to the fact that that the dumped cargo hits the seabed with a considerable impact. Soft material will be whirled up, and the dumped material may also spread over a considerable distance. Nevertheless with the dumping method, it is impossible to create steep under water slopes.

When pumping material, the disposal area may be on land or in water. If it is in water, the sand-water mixture can be discharged at the surface, or alternatively brought down to the bottom via a vertical pipe or a diffuser. In the latter case, it will hit the bottom and form a sort of stilling basin, from which the material settles when it flows over the edge of the "crater". In this way, relatively steep slopes can be created with slopes between 1:5 and 1:7. If material is discharged at the surface, less steep slopes are created. If it is necessary to create slopes, of a specified steepness it will be necessary to construct retention bunds beforehand, or to trim the slopes afterwards.

Disposal on land can take place in both unconfined disposal areas and in diked disposal areas, where the sand water mixture is retained until the particles have settled. Unconfined disposal areas tend to form rather flat slopes, the actual slope depending on the grain size. When bunds are used to contain the dredged material, one must realise that during dredging, water will certainly seep through the bunds, so that the outer slopes will be subject to outflowing ground water. This means (see section 12.1.6) that the equilibrium slope will be reduced from ϕ to ½ϕ, or if ϕ= 30°, a slope of about 1:4. In many cases, an attempt is made to create steeper retention bunds. They are stable only if seepage of water can be prevented, i.e. by applying plastic membrane on the inner side.

Whatever the method of disposal, the dredged material creates a surcharge on the original bottom. Settlements must therefore be expected, and more importantly, there is a serious risk that the subsoil has insufficient bearing capacity and that loss of stability will occur in the form of slips. This risk is greatest immediately after each increase in the surcharge, when the water pressures are increased, and the material has not yet consolidated. In many cases, in order to avoid slips one must introduce waiting times between the deposition of successive layers of fill. Soil mechanical analysis of disposal sites is therefore almost as important as analysis of the dredging area.

If the disposed material is likely to be moved by waves or currents, protection of the slope must be considered. Sometimes there is enough opportunity to place granular filters or a geotextile over the disposed material before serious erosion takes place. In other cases, again retention bunds may be required. These must have sufficient resistance against erosion, to prevent the loss of dredged material.

Improving the permeability, by using sand piles or synthetic fibre drains may enhance the consolidation of the subsoil. A temporary surcharge will also accelerate the settlement.

A special problem is encountered when clay or silty material is pumped into a confined area. The particles will settle slowly with very loose packing, and the dewatering of this sludge takes a long time, making access to the area virtually impossible. The drying out of a crust on top of the sludge will enable access by lightweight vehicles (or persons). Crust formation is enhanced by proper drainage of rainwater. Special remote controlled equipment may be required to create a drainage system. Another technical solution can be achieved by intermittently placing the fine material between thin layers of coarse sand that provide horizontal drainage layers in the mass of impervious material (D'Angremond, 1978).

14. COMMON DREDGING EQUIPMENT

14.1 General

Dredging equipment has been developed by dredging contractors and shipbuilders based on the various principles for disintegration, vertical transport, horizontal transport and deposition. In the design of the equipment, careful attention has been paid to coordinating the capacities of the various processes that take place on board. To obtain insight into common relations those who do not possess detailed knowledge, will find it worthwhile to collect published data on the fleets of the major dredging companies. Analysing the data of the dredges will indicate trends in the size and capacity of parts of specific types of dredges.

Workhorses of the modern dredge fleets are:

- trailing suction hopper dredges
- cutter suction dredges.

For finishing jobs in less accessible places these dredges are sometimes assisted by:

- grab dredges and
- backhoe dredges.

Rather old fashioned, but still used are:

- bucket ladder dredges
- plain suction dredges
- barge unloading dredges

Descriptions and many pictures of these types of dredges can be found in "Dredging for Development", edited by IADC. The description in the following paragraphs supplements the information contained in this book.

14.2 Types of dredges

14.2.1 Trailing suction hopper dredge

The trailing suction hopper dredge is a seagoing vessel. When dredging it tows one or two suction pipes over the seabed. Dredged material enters the suction pipe via the draghead. This draghead shaves thin layers of material from the bottom. The other end of the suction pipe is connected to the hull of the vessel, By a pumping action a mixture of soil and water is pumped into the hold of the ship, the hopper (Dutch: beun). When this hopper is filled with the mixture, it starts overflowing. The excess water flows overboard and the sediment remains largely in the hopper. Loading stops when the carrying capacity is reached (this can be by volume or by tonnage). The load can be discharged either by dumping through bottom doors or by pumping ashore.

Since this type of dredge is an independently sailing vessel, it poses no obstruction to navigation. Moreover, it can work during fairly poor weather conditions. The accuracy of the dredging is moderate since the position of the dragheads cannot actively be controlled.

Hopper volumes ranging from around 1000 m^3 to over 20 000 m^3. This has a direct influence on the weekly production, which may be as high as a million m^3 per week.

14.2.2 Cutter suction dredge

The cutter suction dredge is a stationary dredge. It is a pontoon fitted at one end with a ladder that supports the suction pipe and on the other end with two spuds. The spuds are anchor poles that play an important role in anchoring the hull and moving it forward during the dredging process.

When dredging, the pontoon swings around the central spud (the working spud). During the sideward movement of the suction opening, a crown shaped cutter-head turns in front of the opening and cuts slices of soil into lumps that can enter the suction mouth. The sideward movement is controlled by two winches that are connected to anchors that are positioned on either side of the dredging area. When a cut is completed, the dredge is moved a little forward so that a new cut can be made. This forward step can be achieved by mounting the working spud on a hydraulically actuated spud carriage, or by alternately using the working spud and the auxiliary spud.

Because of the increased power of the cutter-head and of the anchor winches, it is possible to dredge very hard material. When dredging rock, the equipment is subjected to heavy wear .

The disposal of dredged material is mostly done by floating and submerged pipelines. Because of the anchors and pipeline, involved the cutter suction dredge forms an obstruction to navigation.

The cutter suction dredge is capable of dredging quite accurately, both on the seabed and on slopes. It is vulnerable in waves.

The output of a cutter suction dredge can be approximated by looking at the pump power and the diameter of suction and delivery pipelines. The larger dredges have pipelines up to 0.9 m diameter and can produce a weekly output of up to 400 000 m^3 per week.

The capability to dredge hard material can be judged from the power of the cutter engine.

14.2.3 Grab dredge

Little can be said about grab dredges, mainly because they are so simple. In general the capacity is low, and in addition, so is the ability to work in bad weather.

The main advantage is that the grab dredge can reach into corners that are not accessible to more powerful dredges. Horizontal transport is mainly by barge. For this purpose some grab dredges are self-propelled, others need separate barges.

The accuracy of working is moderate. Although the grab can be positioned accurately, it is not possible to finish a horizontal bed An advantage is the almost unrestricted dredging depth.

14.2.4 Backhoe dredge

The backhoe dredge is the modern sister of the grab dredge and the bucket ladder dredge. Hydraulically driven arms control the bucket. Because of the horizontal forces when digging, the pontoon is generally anchored with the aid of spud poles. This makes the dredge rather vulnerable during adverse wave conditions. Transport of material is mostly by separate barge. Dredging accuracy is moderate to good.

The output is relatively low.

14.2.5 Bucket ladder dredge

The bucket ladder dredge used to be the traditional equipment of the Dutch dredging fleet. However, most bucket dredges have been replaced by trailing suction hopper dredges and cutter suction dredges.

The bucket ladder dredge is a pontoon moored on six anchors (one bow anchor, one stern anchor and four side anchors). It swings around the bow anchor that is placed up to 1 km in front of the dredge. While the dredge is swinging, the bucket chain turns round and the buckets at the lower end of the ladder cut themselves full of soil. The full buckets move up along the ladder until they discharge their load as they topple over at the upper end. The soil then flows by gravity along a chute into a barge moored alongside the dredge.

The capacity is low, the dredge is vulnerable in adverse wave conditions, and with its six anchors, the dredge is a nuisance to navigation. However the accuracy is good. The main advantage of the bucket ladder dredge is that it can dredge material without mixing it with water. Therefore the dredge is well suited to dredging the clay that is required for hydraulic structures. The bucket ladder dredge is also still used to dredge material that has been disintegrated by blasting.

14.2.6 Plain suction dredge

The plain suction dredge is again a pontoon equipped with a ladder that supports the suction pipe. Unlike the cutter suction dredge, it has no mechanical tool to disintegrate the soil in front of the suction mouth although most modern plain suction dredges are equipped with powerful jet nozzles.

The dredge is fixed by 4 to 6 anchors and so forms an obstruction to navigation. Because the suction pipe moves around in an area with liquefied sand, the risk that the suction mouth hits the bottom is not too great, and its vulnerability to wave action is moderate.

Discharge of dredged material is mostly by pipeline, sometimes by barge,

This type of dredge leaves a very uneven bottom and usually it is in not used to produce channels or harbour basins. Its main purpose is the winning of sand for reclamation purposes. High production rates can be achieved in clean sand when dredging at great depths is allowed.

The weekly output is comparable to the output of the cutter suction dredge.

14.2.7 Barge unloading dredge

The barge-unloading dredge is used only when traditional barges are used for the horizontal transport of sand, and when the disposal site is not accessible for vessels. The use of a barge-unloading dredge means that material is rehandled. In this case at least three independent processes are combined in one cycle (dredging, sailing transport and pipeline transport). Each of these processes has its workability and its own delays. Therefore, to reduce the downtime of the system careful analysis is required.

The barge unloading dredge pumps water into the transport barge and in this way forms a mixture that can be pumped ashore.

15. COST AND CONTRACTS OF DREDGING PROJECTS

15.1 Cost

15.1.1 General

The cost of a dredging project is often determined by estimating production rates per week for various types of equipment or for various pieces of equipment in a category (Cutter dredge versus Trailing suction hopper dredge, Cutter dredge A versus Cutter dredge B). Then, The cost per week for the equipment is determined, which leads to either a unit price per m^3 of material or a total price for the project.

Methods used to determine production have been discussed; however, methods for the determination of the cost of equipment have not yet been treated.

The weekly cost for a piece of equipment is composed of the following elements:

- Depreciation and interest
- Maintenance and repair
- Labour
- Fuel and lubricants
- Insurance
- Surcharge for company overheads
- Surcharge for profit and risk

15.1.2 Depreciation and interest

Any organisation that invests in dredging equipment will be faced with the payment of interest over the investment cost. When the purchase of equipment is for a one-time activity, the organisation must also recover the cost of that investment in that work, when it is a continuous operation, the organisation needs to replace the equipment after its economic or technical life has ended.

In practice this means that at the beginning (or end) of each financial year a provision has to be made for depreciation and for interest payments. This provision is based on the value of the equipment, whether or not new value or replacement value is meant by this.

Converting this annual charge to a weekly rate that is to be levied when the equipment is working, means that one has to assess the number of effective (i.e. paid) working weeks per annum, averaged over the life time of the equipment.

This calculation leads to endless disputes. To avoid this, the Dutch contractor's association VG Bouw (formerly called NIVAG) has developed certain standard calculations for rates of payment for use by its members. These standards contain an objective method to assess the replacement value of equipment. Based on this value, the weekly rate is determined on the basis of a unified lifetime of the particular piece of equipment. The method is published in a booklet that is updated once in three years ("Operating cost standards for construction equipment"). Basically, the method was developed for contractors, working in joint ventures, or renting equipment from each other on a regular basis. Therefore, the method does not reflect momentarily fluctuations in price level due to market conditions. When the method is used to make budget estimates, correction factors for this influence must be included.

15.1.3 Maintenance and repair

The costs of maintenance and repair are equally disputable as the costs of depreciation and interest. Again, the booklet referred to in section 15.1.2 gives cost standards, based on conditions in the Netherlands. This means that the current costs of spares and of labour in the Netherlands are taken into account. In other countries these costs may differ very greatly. As maintenance and repair are closely connected to wear and tear due to abrasion by sand and rock, the rates are again based on the conditions in the Netherlands, which are very moderate in this respect. For more extreme conditions, corrections will have to be made. In some cases, wear and tear are so dominant with respect to the maintenance cost (e.g. of the pipeline) that the in addition to the rate for maintenance, wear is measured by surveying the equipment before and after use.

15.1.4 Labour

The cost of labour depends on the number of crew on board of the dredge and on the unit rate per week. Much depends on local conditions. When expatriate crew is used, owners of equipment will try to minimise their number because of the effect on the overall cost.

15.1.5 Fuel and lubricants

Fuel consumption depends on the number of operational hours and on the installed horsepower. Operational hours are high with European contractors: around 90% for trailing suction hopper dredges, and around 65% for stationary dredges. Fuel consumption is often expressed in grams or litres of fuel per kW (or hp) per hour. Actual price differs from country to country, often based on whether dredging equipment is seen by the revenue department as ocean going or inland. Usually, marine diesel is the standard type of fuel, but increasingly heavier types of fuel are used because of their lower cost. The added cost of lubricants amounts to 5 to 10% of the cost of fuel.

15.1.6 Insurance

Plant will generally be insured against a variety of of risks. The premium is in the order of 0.1% of the value per week. In some cases, extra premium has to be paid for special risks such as war, fluctuation in exchange rates, political risks).

15.1.7 Overheads

Most dredging companies levy a surcharge of around 8% on their turnover to cover head office expenses. In addition to other expenses these include amongst others the cost of the general management and the cost of unsuccessful tenders.

15.1.8 Profit and risk

Each dredging contract involves a certain amount of risk due to uncertainty about various matters including soil conditions, working conditions and financing,. In normal conditions, contractors usually add a surcharge in the order of 5%. Profit in the order of 5% is usually also charged .

When the contractor feels that a project involves more than the standard risks, he will assess the extra risks and include a provision in his tender offer. One may not expect that the contractor will deliberately underestimate the risk, certainly not if he feels that the employer is transferring risks that are completely beyond the influence of the contractor. Examples of risks beyond the influence of the contractor are:

- siltation during execution of the works
- rate of exchange

- changes in sales tax or cost of fuel
- issuing of certain licenses, etc.

In such cases the employer must carefully consider which risks he wants to transfer to the contractor, and which risks he is willing to bear himself. If the budget estimate by the employer and the lowest offer by a tenderer differ considerably, discussion may reveal that the difference is caused by over-estimation of certain risk aspects in the view of the employer. A solution may be found by re-writing the conditions of contract that give rise to the difference. On the average, an employer will be better off if he does not transfer too many risks to the contractor, unless the contractor has an influence on the matter.

15.1.9 Other cost elements

Apart from the weekly operating cost of the direct equipment, there are charges for the site office, including survey equipment, survey crew, project management, provisional items for the employer.

15.1.10 Review

Cost analysis based on the above techniques will always lead to large deviations from the actual prices quoted. This is caused by the fact that the cost items mentioned in sections 15.1.2 and 15.1.3 constitute a large part of the cost, but do not generally involve direct out of pocket payments by the contractor. These cost elements are credited to an internal account. This means that the contractor may decide to defer the relevant payments to this account if he wishes to increase his chances in the bidding procedure for a certain job. Such situation cannot continue forever, and in better times, extra sums will have to be credited to the internal account, which increases the price level in periods with better market conditions.

15.2 Contracts

Contract conditions differ from country to country. Generally, employers and contractors in any country are accustomed to their national conditions of contract and to the way they are interpreted in case of dispute..

As dredging work is a very international activity and demands a highly specialised type of contracting, the use of standard national conditions of contract does not always lead to satisfactory results. Therefore, FIDIC conditions of contract are often used for international dredging projects. When using those general conditions of contract, one must realise that these conditions impose very strict and specific tasks and roles on the shoulders of employer, engineer and contractor. Though specially written for large international construction contracts, the FIDIC conditions are not always easy to apply to dredging jobs. The International Association of Dredging Companies has therefore produced a number of useful hints for users of the FIDIC conditions of contract (Anonymous, 1990).

Whatever the legal aspects of the contract documents, the employer must think about the technical structure of the contract. What services does he actually want from the contractor, in what way will he measure whether (and in how far) the contractor has fulfilled his obligations. Then, and in what way will he pay the contractor for his direct effort, and possibly give an incentive for extraordinary performance or a penalty for substandard performance. It is essential that contract should reflect the intentions of the employer.

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Appendix 1 HISTORY OF OUR SOLAR SYSTEM

Ingmanson and Wallace (1985) have described the origin of the universe, Earth, ocean, and atmosphere. They think that the universe came into existence between 10 and 20 billion years ago (NB one billion = 10^9!). This estimate is still subject to changing and has been made via three approaches. These approaches are:

- nuclear chronology (based on rates of formation and relative amounts of the elements uranium, thorium, osmium, plutonium, and rhenium);
- studies of the age of the oldest stars;
- measurements of the rate at which the universe has expanded.

According to the model most widely accepted by astronomers, the universe originated in a great explosion, the so-called big bang. This model is consistent with observations first made in 1929 that distant galaxies are receding from the Earth at velocities proportional to their distance from Earth. In 1948 George Gamow predicted that astronomers would one day detect background microwave radiation left over from the big bang. In 1965, Penzias and Wilson proved Gamow right when they detected that radiation, and subsequent measurements provided further confirmation. Other theoretical models have been proposed to explain the origin of the universe, but these have proved deficient when tested against observations and physical measurements.

Although we shall never know all the details of how the sun formed, many astronomers accept the gravitational collapse theory (Figure A1-1). According to this theory, all stars, including the sun, are formed in much the same way, and planets sometimes emerge as a natural by-product of their formation.

Interstellar space contains vast amounts of gas, of which 99% consists of hydrogen and helium atoms. These gases frequently accumulate into more or less coherent clouds or nebulae (Latin for clouds or mist). One such nebula is believed to have collapsed in response to gravity to form our solar system. Its initial mass was probably slightly greater than the present mass of our sun (approximately $2*10^{30}$ kg).

As the nebula contracted, its rate of rotation increased and consequently the nebula began to flatten. It continued to contract until most of the matter had coalesced into a central mass, which ultimately became the sun. A small portion of the nebula survived as a flat disc spinning around the central mass, and it was from the matter contained in that disc that the planets eventually formed.

As the proto-sun (proto- from the Greek for "first, foremost, earliest form of") continued to contract, its internal temperature rose from tens of thousands to several million degrees Kelvin. The immense internal pressure that developed due to particle collisions eventually halted further gravitational contraction, and the sun stabilized. Nuclear fusion, which occurs at such extreme temperatures, released sufficient energy to maintain the temperature and pressure at constant levels, thus stabilizing the sun at essentially the same size as it is now. This whole process of formation, from nebula to stable star, probably required several tens of millions of years and occurred some 4.6 billion years ago.

While the proto-sun was undergoing the final stages of contraction, the flat disc of gas, solids, and liquids spinning around it, was forming into planets. The planets are believed to have grown through a steady process of accretion in which dust particles, molecules, and atoms at first joined together to form larger bodies, which in turn coalesced into larger and larger bodies. In time, through collision and gravitational attraction, these bodies developed into what we call planets. Reasons to regard this scenario as plausible are many. The orbits of the planets lie in roughly the same plane (except Uranus, Figure A1-2), and they revolve around the sun in the same direction and in virtually circular orbits (except Pluto). It seems likely that these highly regular orbital characteristics were established during the collapse of the nebula, before the planets formed.

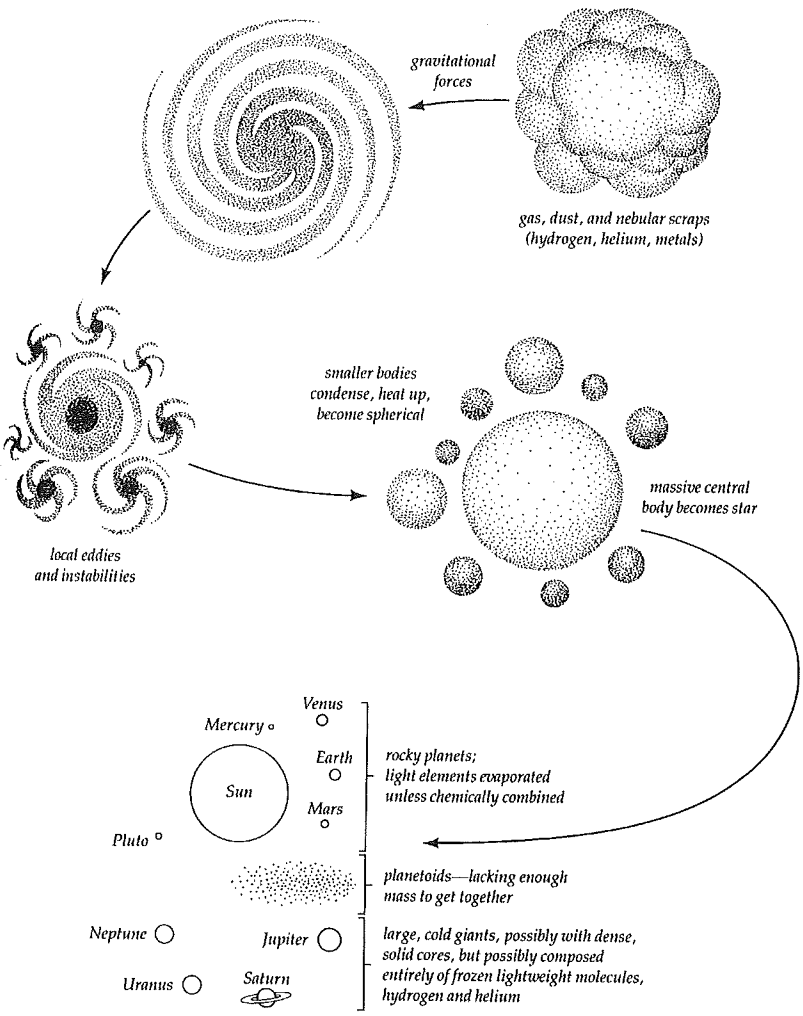
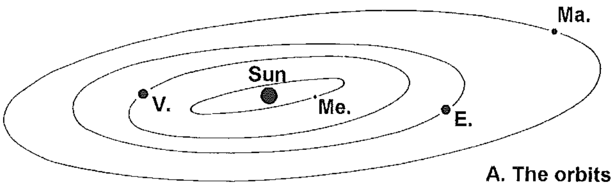


Figure A1-1 Model of the gravitational collapse theory of the origin of the solar system (after Ingmanson and Wallace, 1985)

The third planet out from the evolving sun was the Earth. As it grew in mass, its temperature increased as a result of the energy released by impacts with meteors and the decay of radioactive elements within the planet. Although its temperature never rose to the level needed to initiate nuclear reactions, it did rise high enough to melt the interior. When this happened, heavier elements, such as iron and nickel, were differentiated from lighter elements, such as carbon, and light minerals, such as quartz. The heavier elements formed the Earth's core, and the lighter materials formed the mantle and crust.

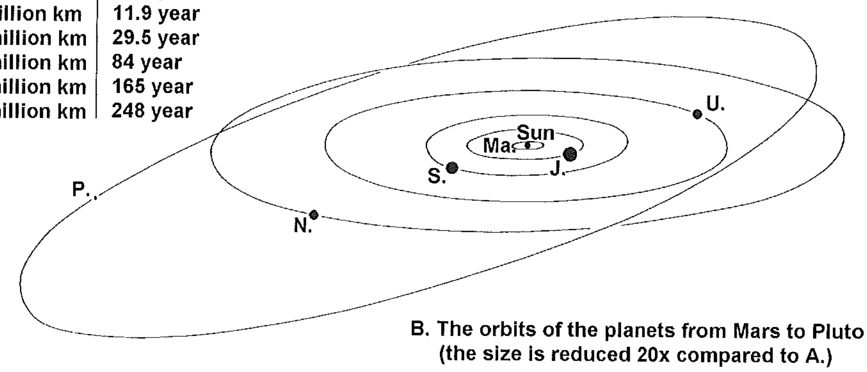
The lightest gases, hydrogen and helium, were too light to be held by the Earth's gravitational field. In fact, in these very early stages of the Earth's history, the gravitational field was probably not strong enough to hold any gases at all. Since the heavier, chemically inert gases (neon, argon, and xenon) are less abundant on the Earth than on other planets, scientists infer that the Earth lost its early atmosphere to space.

Where did the water now contained in the Earth's oceans and atmosphere come from? The answer lies in the assumption that volcanoes were abundant early in the Earth's history and that impacts by meteors caused gases to escape from the Earth's surface. Volcanic gases consist mainly of water vapor, nitrogen gas, and carbon dioxide. If the surface temperature of the early Earth had been about the same as it is now, the water vapour would have condensed to liquid water and the nitrogen gas and carbon dioxide would have formed the atmosphere.



A. The orbits of the planets from the sun to Mars

Planet	Distance to Sun	Circulation Time
Me. Mercury	58 million km	88 days
V. Venus	108 million km	225 days
E. Earth	150 million km	1 year
Ma. Mars	250 million km	1.88 year
J. Jupiter	778 million km	11.9 year
S. Saturn	1580 million km	29.5 year
U. Uranus	2872 million km	84 year
N. Neptune	4500 million km	165 year
P. Pluto	6000 million km	248 year



B. The orbits of the planets from Mars to Pluto
(the size is reduced 20x compared to A.)

Figure A1-2 Planetary orbits around the sun (Grote Bosatlas)

Would the condensation of the water vapor into liquid water have been sufficient to form the oceans? At the present rate of volcanism, the Earth would have to be three times as old as we believe it to be (4.5 billion years) for condensation to have produced the oceans as they exist today. The rate of volcanism may have been considerably greater in the past than it is today; in which case condensation of the water vapor produced by volcanoes might have been sufficient to create the present-day oceans.

Water vapor may also have been released when the impact of meteors raised the surface temperature of the early Earth high enough to melt the outer layers. If the composition of those layers were similar to that of meteorites, which contain about 0.5% water, melting would have released large amounts of water vapor. As time passed, the frequency of impacts would have declined, since the meteors near the Earth would have collided with it early in its history. The Earth would have subsequently cooled, and the water vapor would have condensed, contributing to the formation of the ocean. Volcanic activity has probably continued to increase the volume of water in the ocean. Still, it is not completely clear how the oceans got their present volume.

Appendix 2 RELATIVE MOTION

Copied from J.P. den Hartog "Mechanics"

CHAPTER XVI

54. Introduction.

In all previous statements in this book about displacements, velocities, or accelerations, these quantities were expressed in terms of a coordinate system "at rest." By that, we tacitly meant that the coordinate system is at rest with respect to what Newton called "absolute space," which is the space of the "fixed" stars. Newton's law of the proportionality of force and acceleration is found to agree very well with experiment when the acceleration is referred to a coordinate system "at rest in absolute space." The earth rotates with respect to that absolute space, so that a coordinate system fixed to our earthly surroundings is not strictly at rest, and Newton's laws do not apply quite as well, but for almost all our engineering applications we can say that an earthly coordinate system is sufficiently close to being "at rest." Only for a few devices, of which the gyroscopic ship's compass is the most notable one, does the rotation of the earth become of engineering interest.

In many practical cases a motion can be described more simply in terms of a moving coordinate system than in terms of an absolute one or an earthly one. Take for example the motion of a point on the periphery of a rolling wheel (Fig. 151, page 173)[1]. The path of that point is a cycloid, and the determination of the velocity and acceleration is complicated. If, however, we set up a coordinate system with the origin in the wheel center, moving with it, and with the x axis horizontal and the y axis vertical, then the path of a peripheral point becomes a circle, and its velocity and acceleration appear very much simpler. Or consider a Watt flyball engine governor (Fig. 166, page 193)[1], of which the balls oscillate up and down while rotating. The actual path in space of a ball is very complicated, and the accelerations are difficult to determine. We are very much tempted to place ourselves as observers on the rotating governor spindle and describe the ball motion with respect to the rotating coordinate system. The motion is then a simple up and down oscillation, and the acceleration is easily found. However this acceleration relative to the rotating system is different from that relative to the surroundings at rest, and only the latter acceleration equals F/m. Newton's law is true for coordinate systems at rest; in general it does not hold for moving coordinate systems.

The science of mechanics did not start with engineering, but with astronomy, and naturally the ancient astronomers described their observations in terms of a coordinate system of which the earth was the origin. The paths they found for the planets were awful, hypo- and epicycloids, and it was a great accomplishment when Copernicus (1473-1543) and Kepler (1571-1630) recognized that these paths could be described more simply as ellipses in terms of a coordinate system with the sun as origin. This discovery was one of the starting points for Newton's great work.

In engineering there are many cases where motions with respect to a rotating coordinate system are simpler than those with respect to absolute or terrestrial space. In the counterweights of aircraft engines there are loose pendulous masses, whose motion in absolute space is very complicated indeed, but which only oscillate with respect to a moving coordinate system attached to the crankshaft. The motion of fluid or gas particles in the blades and passages of turbines or rotating pumps are other examples of this kind.

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1 Refers to a non-copied part of the book by Den Hartog

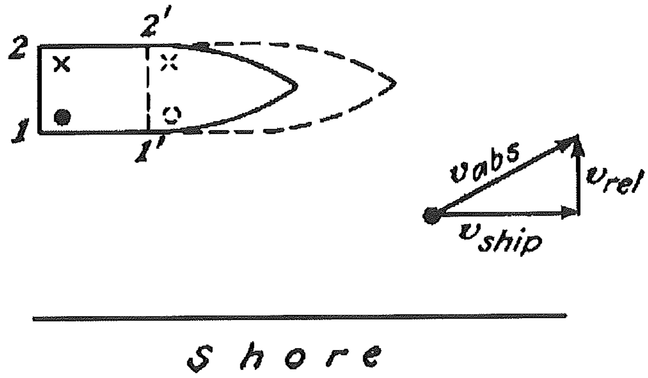


Fig. 261. The absolute velocity is the vector sum of the relative velocity and the vehicle velocity

Thus we recognize the desirability of finding out what we have to do in order to make Newton's laws applicable to moving coordinate systems, and that is the object of this chapter. We shall make no change in Newton's law itself, but we shall find rules by which the actual or absolute acceleration can be deduced from the simpler relative acceleration with respect to the moving coordinate system.

Some of the rules of relative motion are extremely simple, almost obvious, and they have been applied here and there in the previous pages already. Consider, for example, Fig. 261, where a ship moves with respect to the shore, -which, being at rest, is an absolute coordinate system. The captain walks across the deck from starboard to port, from point 1 to point 2. While he does that, point 1 of the ship goes to position 1' and point 2 of the ship goes to 2', and, of course, the captain ends up in position 2'. We call 1-2 the relative displacement and 1-1' the vehicle displacement. The word "vehicle" will be used throughout for the moving coordinate system; in this case the vehicle is a ship; in further examples the vehicle will be a turbine rotor, an elevator cab, a rotating table, the earth, or a crankshaft. In the example of Fig. 261 the captain is the "moving point," which moves relative to the "vehicle" through path 1-2 and relative to "absolute space" through path 1-2'. **By "vehicle" displacement, velocity, or acceleration we shall always mean the displacement or velocity or acceleration of that point of the vehicle which happens to coincide with the moving point at the beginning of the displacement.** This in our case is point 1. The statement here has not much significance because all points of the ship have the same displacement, but in future cases of rotating vehicles it is important to keep this definition in mind.

From Fig. 261 we draw the conclusion that **the absolute displacement is the vector sum of the relative displacement and the vehicle displacement**.

We shall see in the next article that this vector addition of a relative and a vehicle quantity resulting in an absolute quantity holds not only for displacements, but also for velocities. It even holds for accelerations, provided the vehicle does not rotate. But when the vehicle rotates, we shall see that the absolute acceleration of a point is not equal to the vector sum of the relative and vehicle accelerations.

55. Non-rotating Vehicles.

We first investigate the case where the vehicle moves parallel to itself, but not necessarily in a straight path. The path may be curved, but all points of the vehicle move in congruent and parallel paths like the baffler pendulum of Fig. 224. We choose one point of the vehicle for the origin O'

of the moving coordinate system and lay the x' and y' axes fixed in the vehicle (Fig. 262). Then the $O'x'y'$ coordinate system moves parallel to itself in a curved path with the vehicle. Let the distance 1-1' be the distance traveled by the vehicle point 1 in time Δt in its curved path. If Δt be made small the piece of path 1-1' becomes almost straight, and the distance 1-1' can be written as $v_v \Delta t$ where v_v is the average velocity of point 1 of the vehicle during the time Δt.
Similarly the distance 1-2 can be written $v_r \Delta t$ and 1-2' becomes $v_a \Delta t$. We have seen that these displacements satisfy the vector equation

$$v_r \Delta t + v_v \Delta t = v_a \Delta t$$

Now we divide by Δt, and let Δt become zero in the limit, so that the average velocities become trueinstantaneous velocities. This leads to the result

$$v_r + v_v = v_a$$

or in words:
For a non-rotating vehicle the **absolute velocity is the vector sum of the relative and vehicle velocities.**

This sentence is only partly printed in bold-face type, because we shall see later that the statement holds true for rotating vehicles as well, although we have not proved it at this time.

We now proceed to consider accelerations, which are rates of change of velocities. In Fig. 262 the vehicle is shown in two positions, time Δt apart.

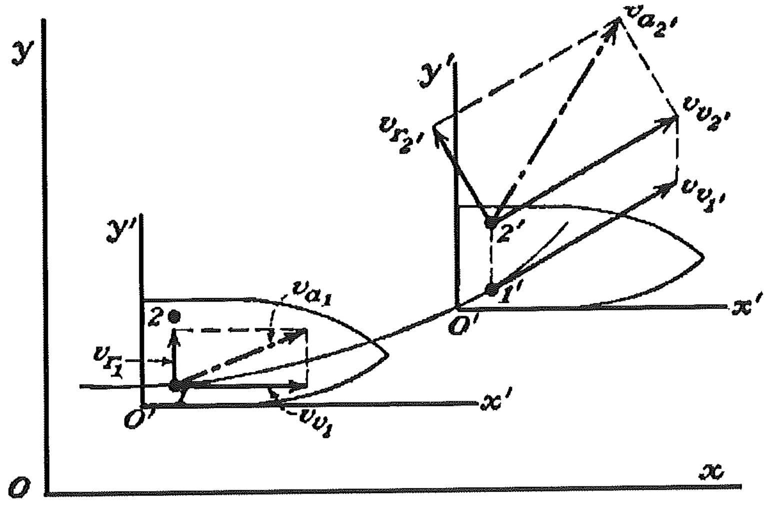


Fig.262. The vehicle and its various velocities shown in two consecutive positions, at t = 0 and at t= Δt

The vehicle is moving through a curved path, and the velocity of its point 1 at the two positions 1 and 1' is different in direction as well as in magnitude. The same is true of the relative speed; at time $t = 0$ the captain walks to portside, but at time $t = \Delta t$ he runs towards the port aft corner of his ship. The two consecutive positions of the "moving point", the captain, are 1 and 2' and the corresponding velocities are plotted in the figure. The vehicle velocity at time Δt is the velocity of point 2' of the vehicle, which is the same as the velocity of point 1' because the vehicle does not rotate. Thus $v_{v2'} = v_{v1'}$. In Fig. 263 the vectors of Fig. 262 have been drawn once more, the velocities at time $t = 0$ in light lines, the velocities at time $t = \Delta t$ in heavy lines, and the differences, which are $\dot{v}\Delta t$, in dashed lines. We see that the directions of the accelerations $\dot{v}$ in general are totally different from the directions of the velocities. From the geometry of Figs. 262 and 263 we deduce that $\dot{v}_a\,\Delta t$ is the vector sum of $\dot{v}_r\,\Delta t$ and $\dot{v}_v\,\Delta t$, by the following process:

$$\begin{array}{rl} & v_{r1} + \Delta v_r = v_{r2'} \\ & v_{v1} + \Delta v_v = v_{v1'} = v_{v2'} \\ \hline & (v_{r1} + v_{v1}) + (\Delta v_r + \Delta v_v) = (v_{r2'} + v_{v2'}) \\ & v_{a1} + (\Delta v_r + \Delta v_v) = v_{a2'} \end{array} \quad +$$

Therefore

$$\Delta v_a = \Delta v_r + \Delta v_v$$
$$\frac{\Delta v_a}{\Delta t} = \frac{\Delta v_r}{\Delta t} + \frac{\Delta v_v}{\Delta t}$$

and

$$\dot{v}_a = \dot{v}_r + \dot{v}_v \tag{31}$$

or in words: **For the case of a non-rotating vehicle the absolute acceleration is the vector sum of the relative and vehicle accelerations.**

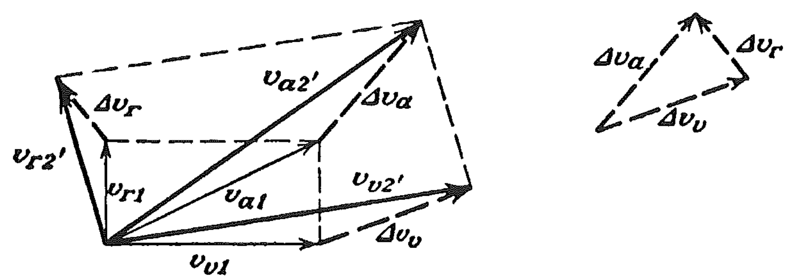


Fig. 263. The velocities of Fig. 262 reassembled into one figure.

Before proceeding, the reader should satisfy himself that the validity of this proof depends on the fact that in Fig. 262 the vehicle velocities of points 1' and 2' are the same. If these velocities are different, which is the case for a rotating vehicle, the formula (31) is false.

Now we are ready to look at Newton's law. It holds only for absolute accelerations:

$$F = m\dot{v}_{abs} = m(\dot{v}_{rel} + \dot{v}_{veh}) \tag{31a}$$

Therefore we may apply Newton's law, and all its consequences of the previous chapters, to the accelerations relative to a non-rotating moving coordinate system, provided we add vectorially to these accelerations the parallel field of accelerations of the moving coordinates.

Before discussing examples of this theorem, we will express it in a somewhat different manner yet. The equation for a moving particle can be written

$$F - m\dot{v}_{veh} = m\dot{v}_{rel} \tag{31b}$$

or in words:

Newton's law may be applied to the relative accelerations of a moving, non-rotating coordinate system, if only we add to each mass element dm a fictitious or supplementary force of magnitude $-\dot{v}_{rel}\, dm$ **.**

As an example consider the space inside an elevator cab rising with an upward acceleration *g/2*. In the cab we, as observers, are looking at a 110-lb lady who stands on a scale, holding a pendulum in one hand and dropping her purse out of the other hand. What is (a) the scale reading, (b) the period of the pendulum, (c) the acceleration of the purse, and (d) the telephone number of the lady?

Applying statement (31a) we observe that the lady has zero acceleration, but we must add $^1/_2$g upward to that before applying Newton's law. The scale thus reads 110 lb for the weight and an additional 55 lb to push the lady up. By statement (31b) we add a force $^1/_2$mg downward to the 110-lb weight of the lady, who thus tips the scale at 165 lb. By statement (31a) the pendulum swings like a pendulum that is accelerated upward at *g*/2, although to me, the observer, no such acceleration is visible. By statement (31b) the pendulum swings under the influence of the gravity force mg plus a fictitious force $^1/_2$*mg* downward. Thus it acts the same way as an ordinary pendulum in a field of $1^1/_2$g = 48.3 ft/sec^2. The purse goes down in absolute space with acceleration g. By statement (31a) we have to add to our observed acceleration an acceleration *g*/2 upward; hence we observe $^3/_2$g downward. By statement (31b) the purse is acted upon by its own weight mg and by an additional downward force $^1/_2$mg; its mass is m, hence it goes down with acceleration $^1/_2$g. Thus questions (a) to (c) have been elucidated. The answer to question (d) is left to the initiative and ingenuity of the reader.

An important conclusion that can be drawn from the theorems (31a) and (31b) is that

Newton's laws apply without any correction to coordinate systems moving at uniform velocity, because the vehicle acceleration f or such a coordinate system is zero.

This places us in a position to clear up the question, discussed on page 233, concerning the applicability of the formula $M = I\ddot{\phi}$ to two- dimensional motion. It was proved that this formula holds for the center of gravity and also for a fixed axis of rotation. We suspected that it might be applicable to the instantaneous center of rotation, i.e., the velocity pole, and possibly also to the acceleration pole. Neither of these two latter points is a fixed center; the velocity pole has acceleration and the acceleration pole has velocity. Let us now view the system from some suitably chosen moving coordinates. First we take a coordinate system moving at uniform speed with the speed of the acceleration pole P_a. Newton's laws apply directly to this vehicle, and with respect to this vehicle the acceleration pole not only has zero acceleration, but zero velocity as well. It therefore is a fixed center, and the **formula** $M = I\ddot{\phi}$ **is applicable to the acceleration pole of a system moving in a plane**.

Next we consider a vehicle with zero velocity but with an acceleration $\ddot{x}_P$ equal to that of the velocity pole. With respect to this coordinate system the velocity pole is a fixed axis, because it has neither velocity nor acceleration. Newton's law is applicable in this coordinate system only after we have added to the system a set of imaginary forces $-\ddot{x}_P\, dm$. If these forces have a moment about the velocity pole they will affect the angular acceleration, and we will find a different answer for $\ddot{\phi}$. If however these supplementary forces have no moment about the velocity pole, we find the correct answer for $\ddot{\phi}$. The supplementary forces are a parallel field $\ddot{x}_P dm$, and their resultant passes through the center of gravity. This force has no moment about the velocity pole if the direction of $\ddot{x}_P$ passes through *G*. **Thus we find that the formula** $M = I\ddot{\phi}$ **is applicable**

to the velocity pole of a two-dimensional motion only when the acceleration vector of that velocity pole passes through the center of gravity.

Another application of the theorems (31a) and (31b) is to the situation inside Jules Verne's projectile traveling to the moon. Looking at this projectile from a terrestrial or absolute coordinate system, we say that the outside shell as well as the passengers and objects inside are all subject to the same acceleration due to the attraction of the earth. Hence everything is floating inside the shell and nothing appears to have weight. Looking at it from a coordinate system moving with the shell, we first establish that the shell has an acceleration towards the earth equal to the local g (which is less than 32.2 ft/sec^2 at some distance). Then we apply to all objects inside the shell the forces mg toward the earth, being the weight, and the supplementary force mg away from the earth. Hence the objects, to an observer inside the shell, behave as if no forces at all were acting on them.

Even when a physical body is rotating, the theory of this article can be applied. The limitation is that the vehicle or coordinate system should not rotate. Consider for example an airplane moving at high speed through a curve, so that the center of gravity has a centripetal acceleration of 5g, while the airplane is turning in space.

We can describe the motion with reference to a vehicle or coordinate system with its origin in the center of gravity of the plane and with its xyz-axes pointing north, west, and up. With respect to this system Newton's laws hold, provided that supplementary forces 5g dm are applied centrifugally. The center of the plane appears at rest, and the plane turns with respect to our coordinate system. It is only when we insist on choosing a coordinate system with the axis directions fixed to the plane instead of to space that the more complicated theory of the next article must be applied.

56. Rotating Vehicles; Coriolis' Law.

When the vehicle translates and rotates, as in Fig. 264, the total or absolute displacement 1-2' can still be considered to be the vector sum of a vehicle displacement 1-1' and a relative displacement l'-2'. Again considering those displacements to take place during the short time Δt and letting Δt go to zero, **the absolute velocity is** seen to be **the vector sum of the relative velocity and the vehicle velocity**, even for the rotating vehicle. In Fig. 264 we could have reversed the procedure, and instead of going from 1 to 2' via 1', we could have gone via 2. Still the above statement holds verbatim, the direction 1-2 is different from 1'-2' and the direction of 1-1' is different from 2-2', but when we go to the limit Δt = 0, all these distances become small, the directions of 1-2 and 1'-2' come closer and closer together and in the limit coincide.

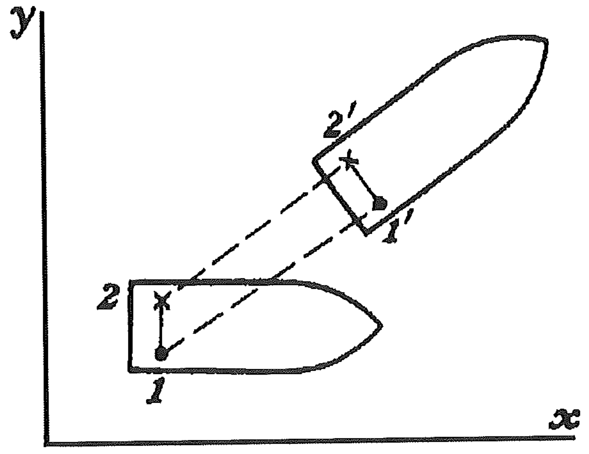


Fig.264. A rotating vehicle

Thus, for velocities, rotating coordinate systems are no more complicated than non-rotating ones, but when we proceed to accelerations we broach a subject that is more difficult than anything we have seen so far in this book. An analytical treatment is apt to hide the physical relations behind mathematical operations. We therefore adopt a geometrical manner of proof, which, although much longer than the analytical one, brings out the physical significance more clearly. The proof will be given for a number of simple special cases, from which the general case will be built up gradually.

We first consider a table rotating at uniform speed ω about its center O (Fig. 265), and a point moving at constant speed v_r, along a radial track attached to the table. The path of that point in space will be a spiral curve and the determination of the acceleration not simple. We now look upon the table as the vehicle, so that the origin of the moving coordinate system O is at rest and the axes rotate. The vehicle acceleration is $\omega^2 r$ toward the center; the relative acceleration is zero, because v_r, is constant. Therefore, if Eq. (31) would apply here, the absolute acceleration would also be $\omega^2 r$ directed centripetally. We can see at once that this is not correct by considering the tangential velocity of our point in absolute space. That velocity is ωr , and it is not constant, because the point moves to larger radii, into a region of greater tangential speed. A point of which the tangential speed increases with time has a tangential acceleration, which Eq. (31) fails to disclose.

Now let us calculate the acceleration of the particle, which is shown again in Fig. 266, in two consecutive positions 1 and 2'. The absolute velocity at point 1 is the vector sum of the relative velocity v_r at point 1 and the vehicle velocity ωr of point 1. The same is true for point 2', but the vehicle velocity there is $\omega(r + \Delta r) = \omega(r + v_r\Delta t)$. The absolute acceleration is the difference between the two absolute velocities divided by Δt. We calculate this difference in components: in the directions parallel to O1-2 and perpendicular to it.

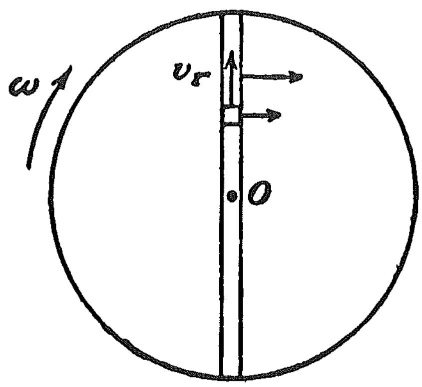


FIG. 265. The first special case of the proof of Coriolis' theorem: a rotating table with a radial track.

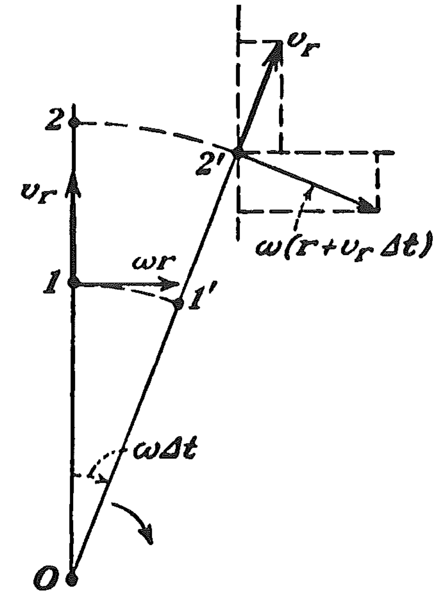


FIG. 266. Toward the proof of Coriolis' theorem. A point moving along a radial track on a rotating table is shown in two consecutive positions 1 and 2′ with all its velocity components.

The angle $\omega\Delta t$ is small, so that $\sin(\omega\Delta t) \cong \omega\Delta t$, and $\cos(\omega\Delta t) \cong 1$, in which terms of the second and higher powers of Δt have been neglected. Then, in the direction parallel to O12, we have

$$\Delta v = \left[v_r - \omega\left(r + v_r\Delta t\right)\omega\Delta t\right] - v_r = -\omega^2 r\Delta t$$

and

$$\frac{\Delta v}{\Delta t} = -\omega^2 r$$

so that

$$\dot{v}_{\text{radial}} = \lim\frac{\Delta v}{\Delta t} = -\omega^2 r$$

In the direction perpendicular to O12 we have

$$\Delta v = \left[\omega\left(r + v_r\Delta t\right) + v_r\,\omega\Delta t\right] - \omega r = \omega v_r\Delta t + v_r\,\omega\Delta t = 2\omega v_r\,\Delta t$$

and

$$\dot{v}_{\text{tang}} = \lim\frac{\Delta v}{\Delta t} = 2\omega v_r$$

The absolute acceleration is thus seen to consist of two components: an outward radial one of magnitude $-\omega^2 r$ (which is a centripetal one of $+\omega^2 r$), and a tangential one to the right of $2\omega v_r$. The first of these is the vehicle acceleration; the second one is something new; it is known as the "Coriolis acceleration," after its inventor Coriolis (1792-1843). Thus, we see that this special case satisfies the following rule:

The absolute acceleration is the vector sum of three components: the relative acceleration, the vehicle acceleration, and the Coriolis acceleration. The Coriolis acceleration has the magnitude $2\omega v_{r\perp}$, where $v_{r\perp}$ is the component of the relative velocity perpendicular to the axis of vehicle rotation. The Coriolis acceleration is directed perpendicular to the v_r vector and also perpendicular to the ω vector of the vehicle.

The rule could have been stated much more simply for this special case; however the reader should verify that, as stated, it is correct for Fig. 266, and we shall presently prove that, in the above form, it applies; to the most general case as well.

The next simple system to be considered is shown in Fig. 267. Again the vehicle or moving coordinate system is a table rotating at uniform speed ω. Instead of the radial track of Fig. 265, we now have a circular track. Let us imagine a toy locomotive running over this track with constant speed v_r while the table rotates.

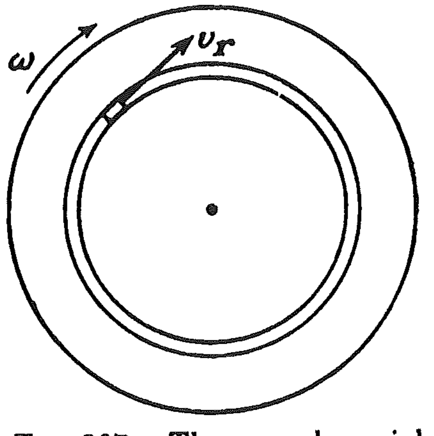


Fig. 267. The second special case in the proof of Coriolis' theorem: a rotating table with a concentric circular track.

Then the absolute velocity of the locomotive is still tangential and equal to ωr + v_r. Its path in absolute space is still the same circle, so that its absolute acceleration is centripetal and of magnitude

$$\dot{v}_a = \frac{1}{r}(\omega r + v_r)^2 = \omega^2 r + 2\omega v_r + \frac{v_r^2}{r}$$

an algebraic sum of three terms, all directed centripetally.

The first of these three terms is seen to be the vehicle acceleration, or the acceleration of that point of the track just under the locomotive. The last term is the relative acceleration of the locomotive with respect to an observer rotating with the vehicle. The middle (Coriolis) term is extra; it has the magnitude $2\omega v_r$, and is directed perpendicular to the relative speed as well as to the ω vector, which, as before, is perpendicular to the table. Thus the result in this case again obeys the general rule.

The third special case to be considered is illustrated in Fig. 268; the rotating table is the same as before, but the track this time is not radial or circular, but perpendicular to the table, and consists of a tube through which the particle is made to move at constant speed v_r. The absolute velocity of the particle in space consists of a vertical component v_r, and a tangential one ωr. Only the latter velocity changes with time and is the same as the velocity of point A of the vehicle. Thus the absolute acceleration is equal to the vehicle acceleration only. There does not seem to be a Coriolis term and, by the general rule, there should not be any, because the relative speed is parallel to the axis of rotation and has no component perpendicular to it. Therefore, this special case also obeys the general rule.

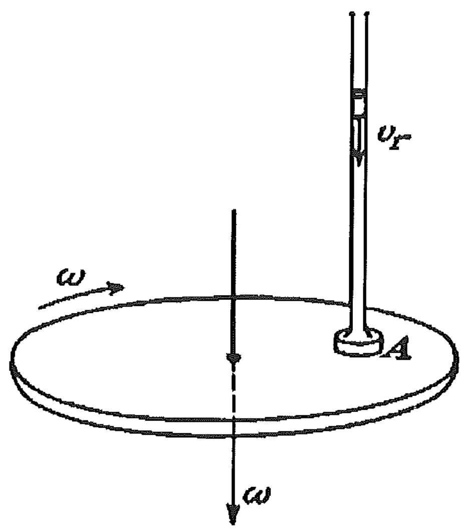


Fig.268. The third special case of Coriolis' theorem: a perpendicular track on a rotating table

The reader should now repeat the reasoning for the three special cases (Figs. 265, 267, and 268), dropping the assumption that ω and v_r, are constants, and introducing the accelerations $\dot{\omega}$ and $\dot{v}_r$, in addition to the velocities ω and v_r. He should verify that the results in all cases conform to the general rule of the previous page.

After the general rule thus has been proved for the three special cases of radial, tangential, and vertical relative velocity, we proceed to a particle of which the velocity relative to the rotating table has all three components simultaneously. The absolute acceleration of that point then in general will have nine components: the relative, vehicle, and Coriolis components for each of the radial, tangential, and vertical cases. The three relative acceleration components add up vectorially to the combined relative acceleration, and the same is the case with the vehicle acceleration. Only for the Coriolis acceleration must we satisfy ourselves that the resultant of the three components still conforms to the rule for magnitude ($2\omega v_{r\perp}$) and direction. Since the case of Fig. 268 has no Coriolis acceleration, and the other two components (for Figs. 265 and 267) lie in the plane of rotation, the resultant Coriolis acceleration is at least perpendicular to the ω vector.

Figure 269 shows in full lines the two components of relative velocity in the plane of rotation, in dotted lines the corresponding Coriolis accelerations. In each case the accelerations contain the common factor 2ω and are further proportional to the v_r component and perpendicular to it. Then the resultant Coriolis acceleration is also perpendicular to the resultant relative velocity and proportional to it, because the dotted rectangle is similar to the fully drawn one.

Thus the general rule of page 9 is proved for the most general case of a rotating vehicle of which the center point is at rest.

For a non-rotating vehicle of which the origin moves, the general rule of page 9 reduces to the special one of page 4, because the Coriolis acceleration is zero. For a vehicle which not only rotates, but of which the origin moves at the same time, we have a superposition of the two previous cases and the general rule of page 9 still holds, although we will not prove it here.

The analytical proof for the two-dimensional case is shorter than the geometrical proof just given. In Fig. 270 let *Oxy* be a coordinate system at rest and *O'x'y'* be a moving coordinate system.

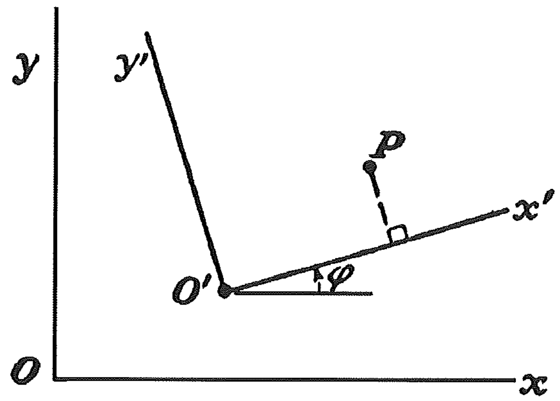


Fig. 270.

A point P has the absolute coordinates x,y and the relative coordinate100s x',y', and the relation between these can be found from geometry :

$$x = x_{0'} + x'\cos\phi - y'\sin\phi$$
$$y = y_{0'} + x'\sin\phi + y'\cos\phi$$

Differentiation gives

$$\dot{x} = \dot{x}_{0'} + \dot{x}'\cos\phi - x'\sin\phi\,\dot{\phi} - \dot{y}'\sin\phi - y'\cos\phi\,\dot{\phi}$$
$$\dot{y} = \dot{y}_{0'} + \dot{x}'\sin\phi + x'\cos\phi\,\dot{\phi} + \dot{y}'\cos\phi - y'\sin\phi\,\dot{\phi}$$

In these expressions $\dot{x}_{0'}$ and $\dot{y}_{0'}$ are the absolute velocities of the moving origin O'; for $\dot{\phi}$, which is the angular speed of the vehicle, we may write ω. Rearranging the terms somewhat, we write

$$\dot{x} = \left[\dot{x}_{0'} - \omega\left(x'\sin\phi + y'\cos\phi\right)\right] + \left(\dot{x}'\cos\phi - \dot{y}'\sin\phi\right)$$
$$\dot{y} = \left[\dot{y}_{0'} + \omega\left(x'\cos\phi - y'\sin\phi\right)\right] + \left(\dot{x}'\sin\phi + \dot{y}'\cos\phi\right)$$

Examining the brackets on the right of the $\dot{x}$ and $\dot{y}$ expressions, we see that they mean the absolute velocities of a point P, when P is fixed with respect to *O'x'y'*. These then are what we have called the "vehicle velocities," by the definition of page 296. The parentheses in the above expressions are the velocities of point P with respect to the coordinates *O'x'y'*. Thus the above equations state in words that the absolute velocity components are the sums of the vehicle and relative velocity components.

Now we differentiate once more. We will do it here only for $\dot{x}$, leaving the similar $\dot{y}$ analysis to the reader.

$$\begin{aligned}\ddot{x} = \ddot{x}_{0'} &- \dot{\omega}x'\sin\phi - \omega\dot{x}'\sin\phi - \omega x'\cos\phi\,\omega - \dot{\omega}y'\cos\phi - \omega\dot{y}'\cos\phi \\ &+ \omega y'\sin\phi\,\omega + \ddot{x}'\cos\phi - \dot{x}'\sin\phi\,\omega - \ddot{y}'\sin\phi - \dot{y}'\cos\phi\,\omega\end{aligned}$$

Rearranging,

$$\begin{aligned}\ddot{x} = &\left[\ddot{x}_{0'} + \omega^2\left(y'\sin\phi - x'\cos\phi\right) - \dot{\omega}\left(x'\sin\phi + y'\cos\phi\right)\right] + \\ &+\left(\ddot{x}'\cos\phi - \ddot{y}'\sin\phi\right) + \\ &-2\omega\left(\dot{x}'\sin\phi + \dot{y}'\cos\phi\right)\end{aligned}$$

An examination of this expression shows that the bracket is the x-component of the absolute acceleration of point P, when P is fixed to *O'x'y'*, and thus the bracket is the vehicle acceleration. The second line is the x-component of the acceleration of P relative to *O'x'y'*, the relative acceleration. The third line upon inspection is seen to be the x-component of the Coriolis acceleration, as defined in the general rule of page 9.

Now we are ready to consider the application of Newton's law to rotating coordinate systems. The law applies only to absolute accelerations, or

$$F = m\dot{v}_a = m\left(\dot{v}_{rel} + \dot{v}_{veh} + \dot{v}_{Cor}\right) \tag{32a}$$

in which the additions must be understood to be in a vectorial sense. This equation can also be written as

$$F - m\dot{v}_{veh} - m\dot{v}_{Cor} = m\dot{v}_{rel} \tag{32b}$$

or in words:

Newton's law applies in a moving coordinate system if only we add to each mass element two fictitious supplementary forces: the vehicle force $-\dot{v}_{veh}dm$, and the Coriolis force $-\dot{v}_{Cor}dm$.

Thus, the term "Coriolis force" means a force equal to the mass times the Coriolis acceleration, directed opposite to that acceleration. It is a fictitious force, of the same nature as the inertia or centrifugal force.

57. Applications

The theory of Coriolis will now be illustrated by applications to

a. Easterly and westerly deviations of projectiles

b. Bending in the arms of a flyball engine governor[4]

c. The man on the turntable

d. The fluid drive of automobiles

a. Easterly and Westerly Deviations of Projectiles.

Imagine a vertical mine shaft a mile deep, located near the equator. If a plumb line is hanging in the shaft and a stone is dropped from rest next to it, the stone will not fall parallel to the plumb line, but will deviate in an easterly direction. The reason for this appears in Fig. 271, which shows the earth when looked down upon from the North Pole. The sun appears to us to run from east to west; hence the earth rotates from west to east.

4 Only example a has been copied here, since it is the only example relevant to the theory of Coriolis applied to the earth rotation

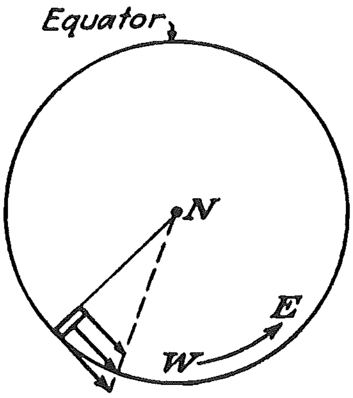


Fig. 271. A stone dropping down a deep mine shaft at the equator.

Looking on the phenomenon from an outside or absolute coordinate system, we see that the stone, before failing, moves easterly with the peripheral speed of the equator. The bottom of the mine pit moves a little slower, being closer to the center of the earth. When the stone is dropped, the only force acting on it is mg downward, so that the (absolute) acceleration in any direction but the downward one is zero. Hence the stone keeps; on going easterly at its original speed and overtakes the bottom of the pit.

An observer on the earth, a rotating vehicle, would reason as follows: If the stone should slide down a purely vertical or radial track parallel to the plumb line, it would go to a region of smaller easterly tangential speed; hence it would experience a Coriolis force to the west from the guide. Since there really is no guide, this force is absent and the stone will deviate towards the east. The Coriolis acceleration is $2\omega v$, where v is the velocity of the stone = gt, and ω is the angular speed of the earth: 2π radians/24 hours. Let the y axis point east, then

$$\ddot{y} = 2\omega g t$$

and integrated twice

$$y = \omega g \frac{t^3}{3} + C_1 t + C_2 = \omega g \frac{t^3}{3}$$

The integration constants C_1 and C_2 are zero because at time $t = 0$ we call $y = 0$ and the easterly speed $\dot{y}$ is also zero. The time t is found from

$$x = \tfrac{1}{2} g t^2$$

where x points downward. Eliminating the time between the two equations, we find

$$y = \frac{\omega g}{3}\left(\frac{2x}{g}\right)^{\frac{3}{2}} \quad or \quad y^2 = \frac{8}{9}\frac{\omega^2}{g}x^3$$

if the depth x = 1 mile = 5,280 ft, and
$\omega = 2\pi / (24 \times 3600)$ radians / sec ,

we find for the easterly deviation
y = 4.6 ft (for x = 1 mile)

If a projectile is shot straight up in the air at the equator, the Coriolis acceleration is opposite to that of the falling stone; the deviation will be westerly, and at the top of the trajectory there will be a westerly velocity. On reaching the earth again there will be a westerly deviation. The calculation is exactly like that of the falling stone, only $-v = v_0 - gt$, instead of $v = gt$.

If a projectile is shot horizontally from a gun at the equator toward the north or south, there is no Coriolis effect. (Why not?) If a projectile is shot at the equator 45 deg upward to the north, the deviation will be westerly, the same as if it were shot purely upward with 0.707 times its initial velocity. (Why?) A bullet fired horizontally toward the north at 45' northern latitude will reach the ground with an easterly deviation.

Appendix 3 THE CLOSURE OF THE AFSLUITDIJK

A3.1 The first steps

The high floods of the late middle ages changed the map of the Netherlands considerably. In many places, the peat formations were eroded and large inland lakes and tidal basins were formed. Peat was also extensively used for the winning of salt and for heating purposes, leaving scars on the landscape. In addition to that, the local rural population had been working continuously to reclaim farmland from the sea by building an infrastructure of dikes and drainage facilities.

From the seventeenth century onwards, it was not only the rural population that took an interest in reclamation and drainage; the rich merchants from Amsterdam also started to invest their capital (gained in the trade to SE Asia) in the development of property for agricultural purposes. In this way, polders were built as commercial ventures in the Province of Noord Holland. The Purmer, Beemster, Schermer and Wormer polders were created this way. The largely artificial drainage of the new farmland relied upon windmills. Hendric Stevin was one of the engineers involved in these works. As early as 1667, he drew up a plan for the reclamation of the Zuiderzee, the tidal basin in the centre of the country. At that time, the plan was 'a bridge too far'. First in the middle of the 19th century, following the invention of the steam engine, which provided a reliable source of power for larger pumping stations, the Haarlemmermeer was reclaimed. From then onwards, a succession of proposals was published (See figures 1 to 4):

1. Kloppenburg and Paddegon (1848)
2. Van Diggelen (1849)
3. Beijerinck (1866)
4. Kooy (1870)
5. Opperdoes Alewijn (1866-1873)
6. Stieltjes (1870-1873)
7. Leemans (1875-1877)
8. Wenmaekers (1863-1883)
9. Buma (1882-1883)

Some of these proposals left the mouth of the River IJssel open, while others included reclamation of the entire Waddenzee. The status of the proposals was also different. Some were developed on the basis of private initiative, others at the request of the Government that considered the project too wide-ranging to be executed by private parties. In spite of this, the Administration could not come to a proposal that was acceptable to Parliament.

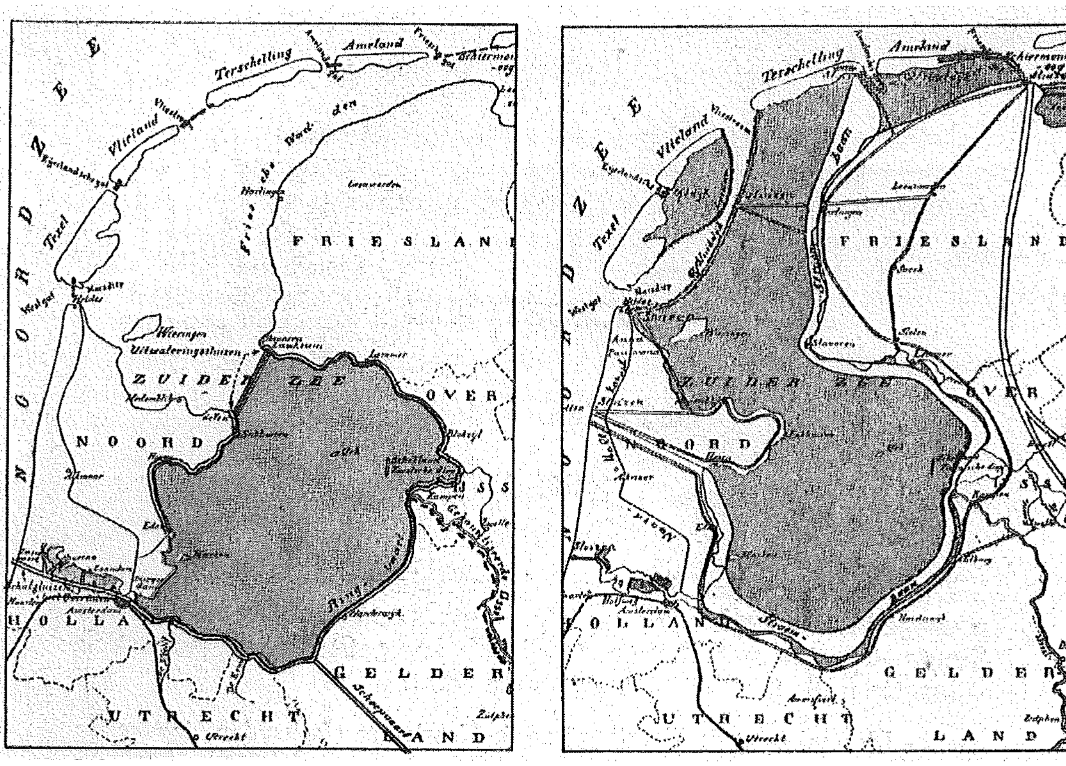


Figure A3-1 Kloppenburg and Paddegon, 1848 (left) and Van Diggelen, 1849 (right)

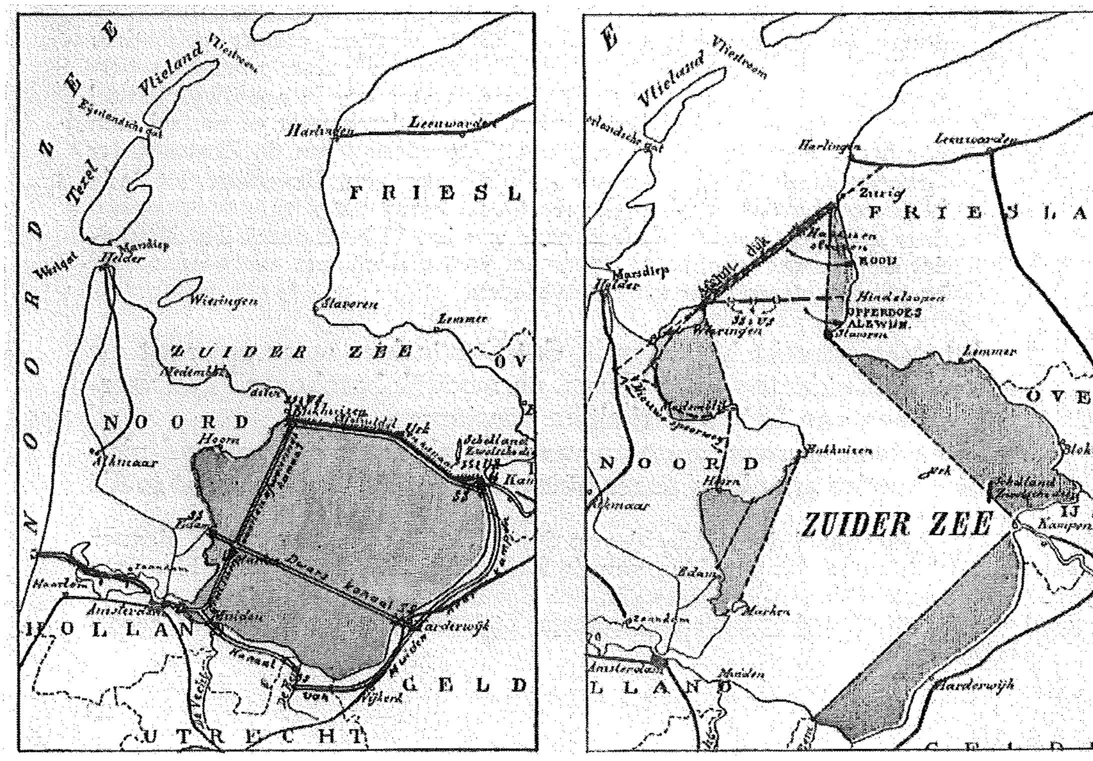


Figure A3-2 Beijerinck, 1866 (left), Kooy, 1870 and Opperdoes Alewijn, 1866-1873 (right)

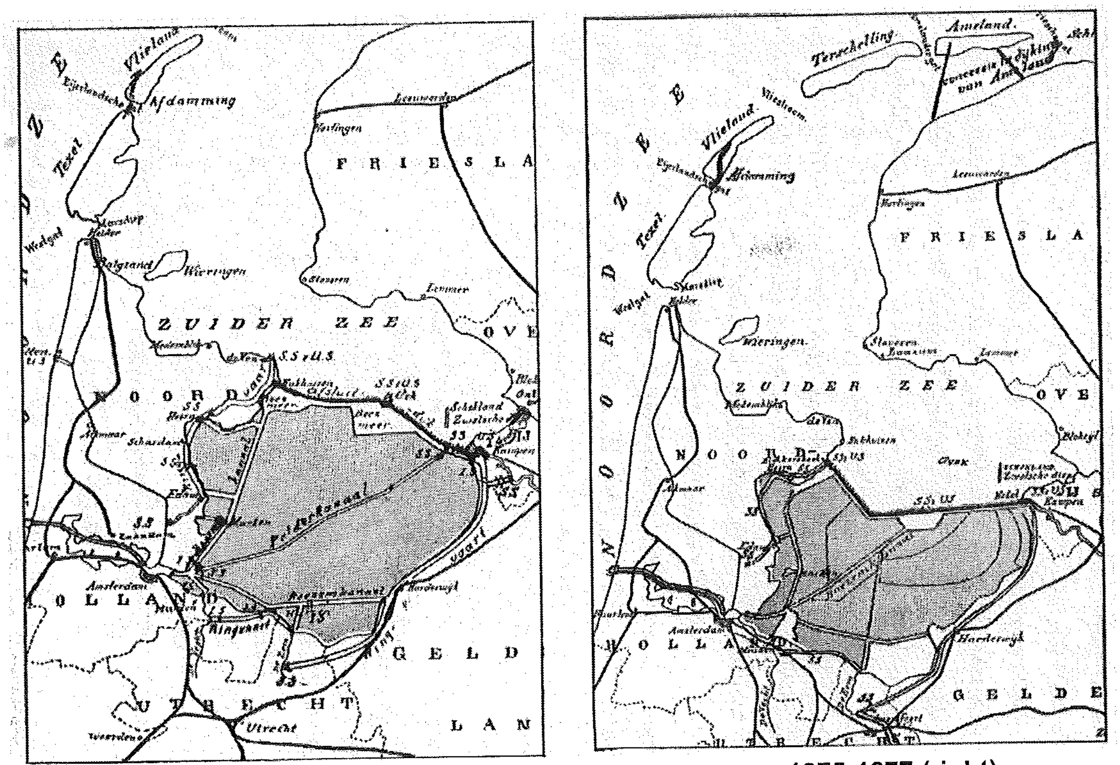


Figure A3-3 Stieltjes, 1870-1873 (left) and Leemans, 1875-1877 (right)

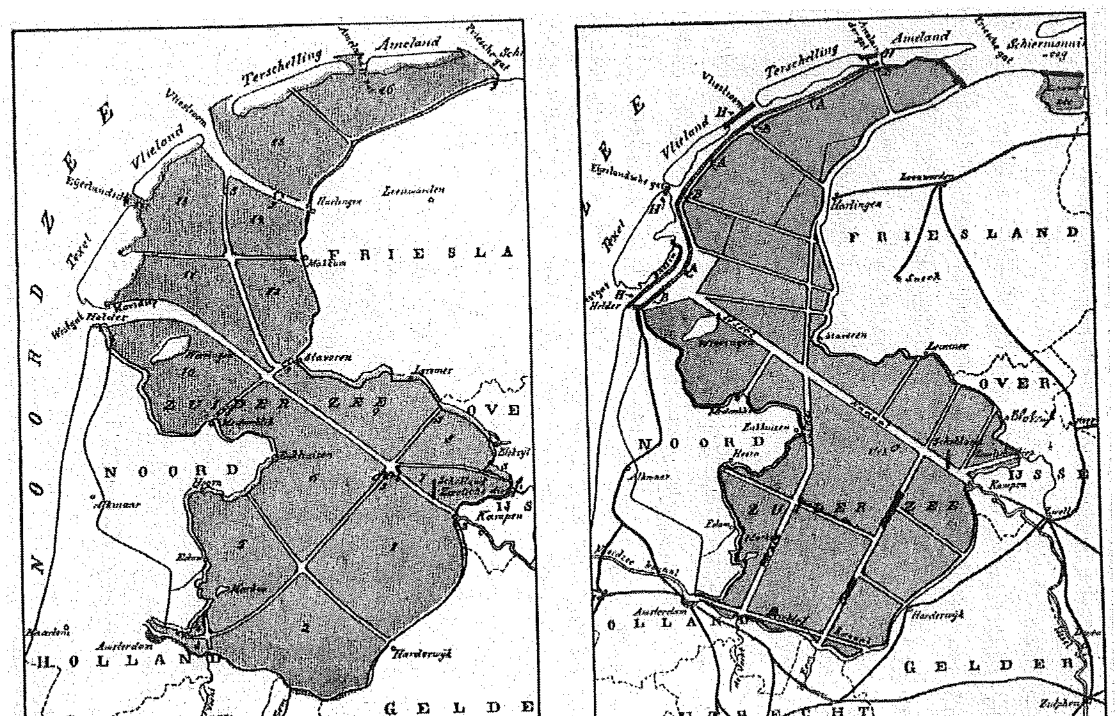


Figure A3-4 Wenmaekers, 1863-1883 (left) and Buma, 1882-1883 (right)

A3.2 Lely and the Zuiderzeevereniging

The interested individuals, kept alive the idea for a closure and commercial reclamation of the Zuiderzee and they established the Zuiderzeevereniging (Zuiderzee Association). This was a private association established in 1886 for the purpose of studying the technical and economic feasibility of a closure and partial or complete reclamation of the Zuiderzee, the Waddenzee and the Lauwerszee. Members of the association were individuals, politicians and representatives of provinces and municipalities. It was quite difficult to raise the funds required for the studies, but by the end of 1886, the Association could appoint two engineers: ir. J. Van der Toorn, a senior engineer from Rijkswaterstaat and the much younger ir. C. Lely. Soon, Van der Toorn resigned from the function, to leave the supervision of the studies to Lely (see Figure A3-5).

Figure A3-5 Ir. C. Lely at older age

The studies were successfully carried out, and after a number of interim reports, the final report appeared in 1891. It concluded that in the North, the soil conditions of the Wadden Sea were unsuitable for agriculture (too sandy). It also indicated that south of the present location of the Afsluitdijk, some areas with suitable clay deposits were present. Lely projected polders in those areas. It was a pleasant coincidence that these areas also formed the shallowest part of the Zuiderzee, so that reclamation was relatively easy. Lely recommended closing off the Zuiderzee first and reclamation of the agricultural land within the protection provided by an enclosing dike. He found that this was a cheaper solution since the polders could then have lower dikes to protect them against storm floods. He also considered the workability would be a lot better if the basin was shut off first.

As to the exact location of the closing dike, the report indicated that the tidal inlet near Den Helder was too deep to close. This being so, it would be logical to connect the closing dike to the shore of Noord Holland in the vicinity of the island Wieringen. The report also considers whether the River IJssel should be permitted to discharge into the newly formed basin, or to construct the closing dam in such a position that the river could still flow into the open sea. On the basis of observations it was concluded that the sediment carried by the IJssel would not pose a problem for the basin, on the contrary, the fresh water would be an asset. Furthermore the new inland lake would protect the coastline of the land around the lake and provide better drainage facilities. Lakes between the new polders and the historic land should prevent an unwanted lowering of the ground water table in the old land. The option to construct a railway line from Amsterdam to Leeuwarden via the enclosing dike was mentioned. The only disadvantage that was foreseen was the damage to the till then flourishing fisheries in the Zuiderzee. The report concluded that the cost of the reclaimed land would be in the order of Dfl 1000 per hectare, which was considered very reasonable.

Summarising, the anticipated advantages were:
- Creation of good quality farmland
- Improvement of the protection against flooding
- Enhancement of traffic connections
- Creation of a fresh water basin

and the disadvantages:
- loss of fisheries

Just before the completion of the final report in 1891, Lely was invited to become Minister of Public Works and Economic Affairs. It is beyond the scope of this book to give details of the political position of Lely and the Liberal party, in which Lely belonged to the radical wing that supported political and social change. As a Minister, Lely could not find enough support for the plans he had drawn up himself. Instead, he decided to install a State Committee to review the plan of the Zuiderzeevereniging. In 1894, the State Committee published its final report that was largely very positive and mentioned additional advantages even greater than those mentioned by the Zuiderzeevereniging. Political complications within the cabinet caused a political crisis, the cabinet resigned and Lely became ordinary Member of Parliament from 1894 until 1897. From 1897 until 1901, was Lely again Minister of Public Works and Economic Affairs, but other items required more attention than the closure of the Zuiderzee. In this period, the Staatsmijnen were established, and important decisions were taken on the improvement of the Noordzeekanaal and the construction of a fishing port at Scheveningen. Although Lely did prepare the Zuiderzee-law, the work finished too late to pass parliament before the elections of 1901. The liberals lost the elections of 1901, and the new Minister of Public Works was a declared opponent of closure. Lely continues his career as Governor of Surinam (1902-1905) and was re-elected to parliament in 1905. He retained this post until 1913 and combined it with the position of Alderman of The Hague from 1908 to 1913.

Political turmoil and the high cost of the complete plan prevented a favourable decision for many years. An attempt was made to split the project by building first one new polder (Wieringermeer) and building the closing dike at a later stage. This would reduce the initial cost., but lose the advantage of polder construction in a calm (non-tidal) environment. It is worth mentioning that in 1907 Lely received an honorary doctorate from Delft University (then still the Technische Hogeschool). Finally, in 1913, Lely became Minister of Public Works for the third time. It was a difficult period. The First World War had broken out and the Netherlands successfully attempted to maintain a neutral position. This meant that the country had to be self-sufficient in food production, which demonstrated the need for increased agricultural production. In the same period, a storm surge struck the northern part of the country and large parts of the province of Noord Holland were inundated. It is no surprise that the Zuiderzeevereniging pointed at the fact that an open Zuiderzee imposed a tremendous risk on the safety of the heart of the country. Eventually, the cabinet agreed with the Zuiderzee Law, which was submitted for parliamentary approval in September 1916. This approval was finally obtained in spring1918, more than 30 years after the founding of the Zuiderzeevereniging. The approved plan was quite similar to the first draft design by Lely from 1892.

A3.3 Final preparations

A separate agency was set up for the design and execution of the works: the "Dienst Zuiderzeewerken", which had close ties with, but was formally independent of Rijkswaterstaat. The works were to be supervised by the Zuiderzee Council of which Lely remained chairman until his death in 1929. Before the actual start of the works, a new State Committee, this time chaired by the Nobel prize winner in physics, Prof. Lorentz was charged with the conduction of a study into the effects of the closure on the future tide levels and storm surge levels north of the Afsluitdijk. Lorentz developed a mathematical technique for tidal calculations based on linearisation of the quadratic terms in the equation of motion, a method that remained in use for many years. The results were unexpected in the sense that the calculations showed an increase in the current velocities in the tidal inlets to the Wadden Sea, whereas a decrease was expected. After completion of the works, the predictions by Lorentz proved surprisingly accurate.

The progress of the works was slow. Soon after the start on the west side of the closure dam, the economic conditions deteriorated, and the works were suspended. Only those activities that served to protect partially completed sections of the work could be continued, which meant that in fact only the dike connecting the island of Wieringen to the mainland was built. This situation lasted until 1925, when again a decision was taken to go ahead. In 1926, a contract was signed with a joint venture of four of the largest contractors in the country, the Maatschappij tot Uitvoering der Zuiderzeewerken (MUZ). The MUZ also won the contract for the polder dike around the first polder in Lake IJssel: the Wieringermeerpolder. Partners in the MUZ were:

- Van Hattum en Blankevoort (later Stevin Group and the still working Company of Volker Wessels Stevin)
- Hollandsche Aanneming Maatschappij (working company of HBG)
- Bos
- L. Volker. (later Adriaan Volker and subsequently Volker Wessels Stevin)

During the final design of the closure dam, many problems arose that were difficult to solve. These included determination of the required width of the discharge sluices in the closure dam, the need for scour protection near the sluices and the need for scour protection in the closure gaps. For many of these problems, reference had to be made to laboratories in Germany (specifically at Karlsruhe) that had just developed the technique of making hydraulic scale models. One of the secretaries of the Lorentz Committee, Thijsse, was charged with the monitoring of these tests. After his return, he recommended that a similar laboratory should be built in the Netherlands, and he founded this Laboratory in 1927 in the basement of the former Department of Civil Engineering of Delft University. Later this laboratory was privatised to become "WL| Delft Hydraulics".

A3.4 The actual works

The works were essentially carried out by using traditional techniques and materials. Clay was dredged in the vicinity with bucket ladder dredges and transported to the dam by towed barges. At the work front, steam driven grab cranes discharged the barges, building a solid clay dam from the seabed to a level well above HW. To support the clay dam, sand was pumped into the final cross section as well.

The clay dredged in the region mainly consisted of over-consolidated moraine material called boulder clay ("keileem"). This material was extremely resistant to currents and with, our present knowledge, we can say that the works would have failed if such good quality of clay had not been available. Scour protection works were made by using fascine mattresses (zinkstukken) made of osiers (willow twigs).

In January 1930, the dike around the NW polder (Wieringermeer) was closed and pumping out the water was basically finished in the summer of 1930. That marked the start of a phase of cultivation: turning the seabed into farmland land. The plan of the new polder was based on scientific social and economic analysis.

In 1932, the Afsluitdijk was closed. The reclamation of the NE polder (Noordoostpolder) continued slowly during World War II, giving shelter to many refugees and members of the resistance. It was envisaged that the sequence of land reclamation should continue with the reclamation of the SE polder (now named Flevoland). Because of the tremendous size of this polder, it was decided to split it in two parts, building a separation dam from Harderwijk to the present location of Lelystad. The Eastern part was completed in 1957. At that time there was a discussion about the sequence of the remaining works. It was considered advantageous to construct the SW polder (Markerwaard) first, so that the remaining part of the SE polder could be constructed in its lee. Actually, a start was made with the dikes around the Markerwaard, but in 1959, for reasons of spatial planning it was decided that Flevoland should be completed first. The Southern part of Flevoland could thus help and reduce the need for space in the region between Amsterdam and Amersfoort. The change of plans can clearly be seen in the landscape: the dike on the NW side of Flevoland (from Lelystad to Amsterdam) consists partly of the dike that was meant to surround the Markerwaard. The locks at Lelystad (Houtribsluizen) were also completed, anticipating the creation of an inland waterway from Amsterdam to the open part of Lake IJssel. After completion of S. Flevoland in 1968, the priorities had changed so much that the Markerwaard was not reclaimed at all. The dike from Enkhuizen to Lelystad was still built, the purpose being to improve the water management in Lake IJssel rather than reclamation of the polder. From time to time the discussion about the Markerwaard and its reclamation is resumed, usually in connection with finding a new location for Amsterdam Airport.

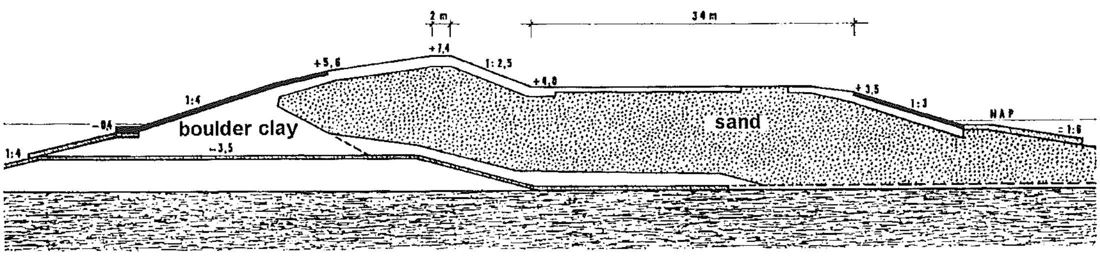


Figure A3-6 Cross-section of the Afsluitdijk

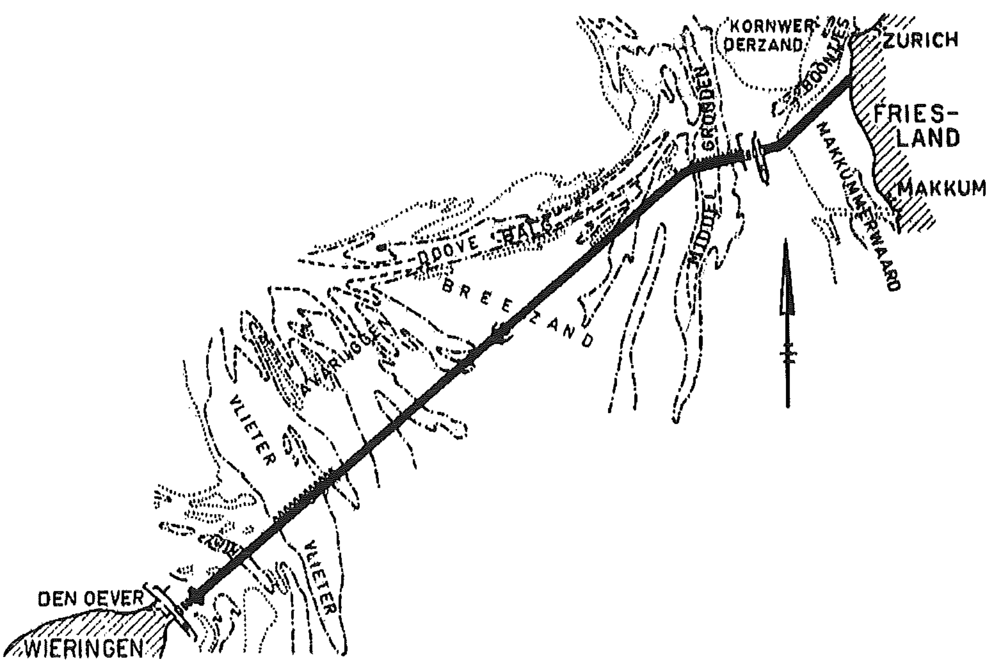


Figure A3-7 Trajectory of the Afsluitdijk

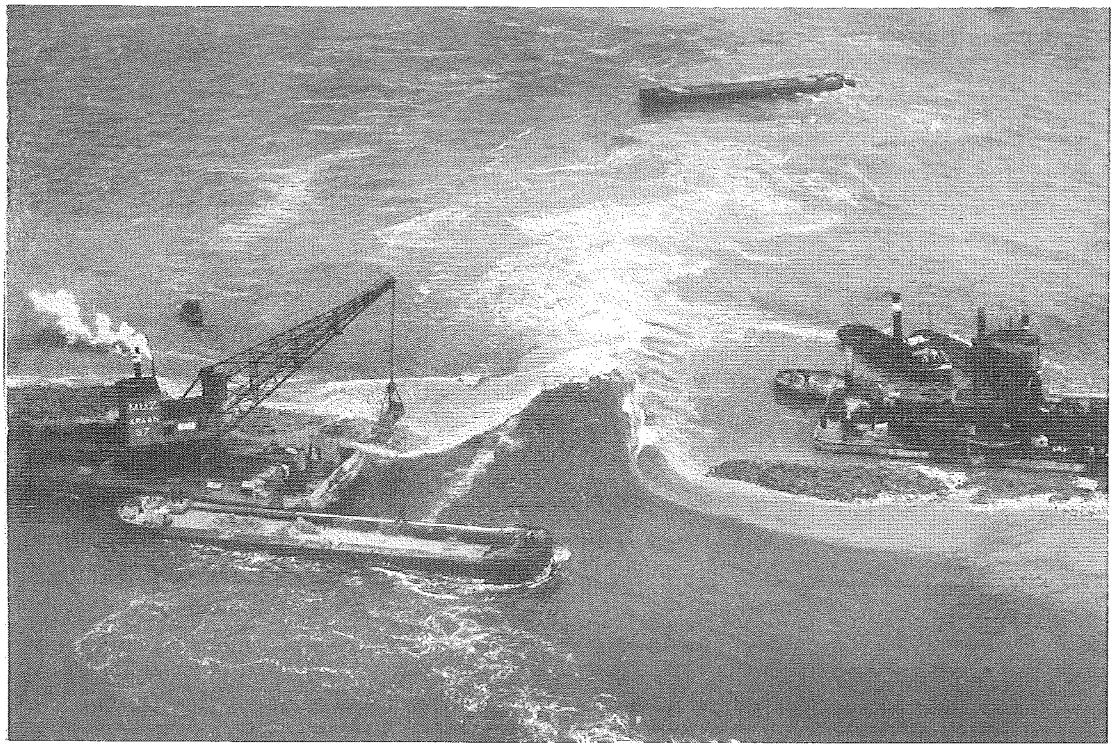


Figure A3-8 Closure Afsluitdijk with Boulder Clay

Some facts and figures have been summarised in Table A3-1. They were related to other closure works much later (see Table A4-1).

Polder		Area (ha)	Period of construction
Planning stage Name	Present name		
NW polder	Wieringermeer	20,000	1927-1930
NE polder	Noordoostpolder	48,000	1937-1942
SE polder (eastern part)	Oostelijk Flevoland	54,000	1950-1957
SE polder (western part)	Zuidelijk Flevoland	43,000	1959-1968
NW polder	Markerwaard	56,000	n.a.

Closing dam	Basin Area	Tidal range	Period of construction
Afsluitdijk	350,000	1 - 1.5 m	1927-1932

Table A3-1 Facts and Figures concerning the Afsluitdijk (see also Table A4-1)

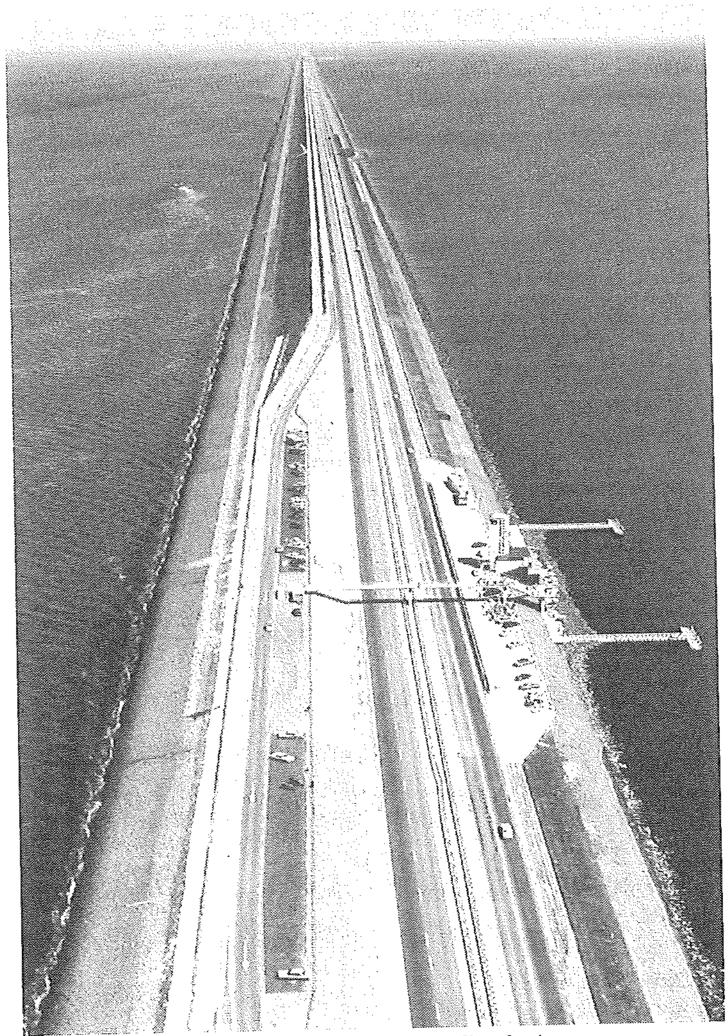


Figure A3-9 Afsluitdijk, final stage

A3.5 The works in hindsight

If the plans of 1891 are compared to the final layout as built, it is remarkable how well these design studies were carried out. With respect to the anticipated advantages and disadvantages in hindsight, it can be concluded that the project had a tremendous positive influence on safety and on the traffic connections. Due to global economic developments, the creation of farmland is no longer considered so important, although the space created has been used to solve many planning dilemmas in the country. The availability of a large fresh water basin has become more important than was ever foreseen. Eventually, the fishing industry did not suffer as much as was feared, because fisheries continued in the fresh-water lake. Completely unforeseen was the present role of Lake IJssel for recreation. It is interesting to see how technical developments in mobility of people (bike - motor bike - car) influence the spatial planning concepts in the various polders.

From a scientific point of view, the spin-off has also been very extensive. The project triggered the start of scientific analysis of coastal and hydraulic engineering in the Netherlands. The method of developed by Lorentz for solving the equations of motion has already been mentioned, as also has establishment of the Waterloopkundig laboratorium (WL| Delft Hydraulics). Furthermore, the preparation of the reclaimed seabed for agriculture has led to comprehensive agricultural research and much insight has been gained into desalinisation. Without the experience of constructing the Afsluitdijk, it would have been very difficult to repair the dikes in the SW part of the country after the bombing of 1944 and the storm surge flood of 1953.

Appendix 4 DELTA PROJECT

A4.1 History

During the night of 31 January to 1 February 1953, a severe storm passed over the North Sea and caused havoc along the coasts of the southern North Sea. In the Netherlands, over 1800 people died and large areas (1340 km^2) of the SW part of the country were flooded. At some locations including the vulnerable dike of the Hollandsche IJssel that protects the heart of the country, breaching of the dikes could just be prevented,

After the rescuing of the survivors, the first priority was to repair the dikes to prevent further damage during an extended inundation by salt water. Specifically, repair of the dikes around the island of Schouwen-Duiveland was a tough job. Although use could be made of equipment and experience developed for the closure of the Zuiderzee there were still some big differences in the problems encounted. The tidal amplitude in the SW part of the Netherlands is considerably greater than that in the North, and moreover, the clay in the South is less resistant to erosion than the Boulder clay (keileem) available in the North. This was known because the same factors had played a role when damage to dikes of Walcheren, destroyed during bombing in World War II was being repaired. Eventually, with the aid of some improvisation, caissons left from the Mulberry Harbour that had been used in Normandy were used in Walcheren . During the remedial works of 1953, such caissons were also used, but by then in a more controlled way. The lessons from 1945 had been learned. Moreover, shortly before 1953, two small inlets had been closed by using new techniques: the Braakman and the Brielse Maas.

The disaster did not come as a surprise to the experts. Wemelsfelder, an engineer with Rijkswaterstaat had studied the statistical behaviour of HW levels observed during over 100 years. He had concluded that the probability of the occurrence of a storm surge higher than most dikes was quite high. Although his work was published in "De Ingenieur"(nr. 9, 1939), no action was taken. Only some ideas were developed by Van Veen to connect islands in the South. This is understandable in the historical context. World War II was just over, and a lot of energy went into the repair of the war damage. Moreover, the Cold War had just started, and all resources were used to improve the defence against the military threat from the communist countries.

A4.2 Design of the Delta Project

The storm disaster may not in itself have been entirely unexpected, and thereafter, swift action was taken. A few weeks after the extensive flooding, the Delta Committee was established, its tasks being to advise the Minister about measures to be taken in connection with the disaster and to indicate the water levels and other boundary conditions that should be used when upgrading the coastal defences in the entire country. Closure of tidal inlets was not excluded, although it was accepted that the entrances to the ports of Rotterdam and Antwerp should remain open.

There were two main points of discussion. The first concerned the level to which the sea defences should be raised. The second major consideration was whether existing dikes should be strengthened rather than building new dams across in the mouths of the tidal inlets and thus considerably reducing the length of the coastline. Furthermore, it was indicated that the plans should also indicate measures against salt intrusion in the SW part of the country.

During its work, the Committee soon reached the conclusion that some urgent steps could be taken independently of the completion of the study. For that reason, some interim reports were made the most important of which were:

May 1953	Construction of a storm surge barrier in the Hollandsche IJssel
February 1954	Closure of the tidal inlets (Delta Project)
January 1955	Three Island Plan (Closure of Veerse Gat and Zandkreek).

Although the final report did not appear before 1960, the interim reports formed the basis for the legislation comprising the closure of the inlets in the SW part of the country. The Delta Law passed through Parliament (Tweede Kamer) on November 5, 1957, and was published on May 8, 1958.

The final report appeared in December 1960, after a period of intensive and comprehensive research. It concluded that the protection against flooding was too low along the entire coastline of the country, with special emphasis on the conditions in central Holland, along the Hollandsche IJssel. On the basis of statistical considerations, the Committee indicated that it considered a storm surge level of NAP +5 m in the Hook of Holland a reasonable basis for a detailed discussion about the safety of the country. The probability of exceedance of this level was estimated to be in the order of 10^{-4} per annum (see Figure 4-10). On the basis of hydraulic and statistical data, equivalent levels were fixed for other locations along the coast.

The report also indicated that calculations had been made to compare the cost of protective measures and the damage caused by flooding. Although no reliable results were obtained from these calculations, it was clear that levels much lower than the equivalent of NAP + 5 m in Hook of Holland would be far below an economic optimum. It was eventually recommended that for the densely populated area between the Hook of Holland and Den Helder a frequency of exceedance of 1/10000 should be used. It was considered that for the less populated and industrialised areas in the North and the South a probability of 1/4000 would suffice. The Committee stipulated that a moderate exceedence of the design level should not immediately lead to a major disaster. (From informal sources one often hears that the economic calculations indicated a higher optimum dike level, but that the politicians of those days did not expect to find a majority for more drastic proposals.)

The report of the Committee then continued with a discussion between the merits of closure of the tidal inlets versus strengthening existing dikes and seawalls. It is recommended closing the inlets in the SW except the Rotterdam Waterway and the Western Scheldt. The main reasons for this recommendation were that it would entail the construction of a modern short and strong coastline and the creation of fresh water reservoirs. An additional advantage would be the improved traffic infrastructure. The Committee also pointed at some side effects on fisheries, the environment and the social infrastructure in the area. (Certain parts of the provinces of Zeeland and Zuid Holland were really isolated at that time.)

In concrete terms, the Committee recommended the closure of the primary inlets (from N to S) Haringvliet, Brouwershavense Gat, Eastern Scheldt and Veerse Gat. The latter closure had already been mentioned in an interim report. Since there would be a considerable time gap between closures of the inlets, secondary dams would have to be built to avoid shortcuts through the channels at the upstream end. This required secondary dams in the Volkerak, Grevelingen and Zandkreek. The closure dam in the Haringvliet had to be equipped with a discharge sluice of sufficient capacity to cope with the discharge of the River Rhine. Remaining dikes that would not be protected by the closure works were to be strengthened to the required level.

The report also pays attention to the possibilities of improved management of the fresh water resources in the SW part of the country. Control of the Haringvliet sluices would give the possibility to increase the discharge via the Rotterdam Waterway, reducing the salt intrusion caused by the continuous deepening of this channel for navigational purposes. A network of inlet sluices was designed to flush the reservoirs of Grevelingen and Eastern Scheldt with fresh

(Rhine) water. Last but not least, new lock complexes in the Volkerak Dam and at the Kreekrak would facilitate the inland fairway from Rotterdam to Antwerp, honouring an already long overdue commitment to Belgium.

The report then discusses the technical feasibility of the works. A comprehensive comparison is made with similar earlier works (see Table A4-1).

work	**time**	**average tide difference**	**cross section of gap below NAP [m²]**	**average tide volume [10⁶ m²]**
Afsluitdijk (closure dam) Zuiderzee				
original profile	1925	0.9	120 000	575
Vlieter + Blinde Geul + Middelgronden	May 1931	1.1	20 000	475
Vlieter + Middelgronden	Oct. 1931	1.2	15 000	375
Vlieter, with dam	Dec. 1931	1.3	10 000	300
Vlieter, shortly before closure	May 1932	1.5	500	20
Walcheren				
Nolle (Vlissingen), largest extension	June 1945	3.7	750	11
Westkapelle, before closure	Oct. 1945	3.3	225	5
Vrouwenpolder (Vere), before closure	Oct. 1945	3.0	300	13
Rammekens, before closure	Jan. 1946	3.8	600	25
Brielse Maas				
original profile	1948	1.8	2 700	17
before positioning of the closure pontoon (Botlek closed)	July 1950	1.8	300	12
Braakman				
before positioning last but one pontoon	July 1952	4.0	850	17
before positioning closure pontoon	July 1952	4.0	350	14
dike restoration				
Kruiningen	June 1953	4.4	550	26
Schelphoek, during largest extension of the gap	May 1953	2.8	8 000	130
Schelphoek, Klompegeul and Gemene Geul	July 1953	2.8	2 000	80
Ouwerkerk, before positioning last two pontoons	Nov. 1953	3.0	1 500	36
Ouwerkerk, before positioning last pontoon	Nov. 1953	3.0	750	33
Delta works (original profile)				
Veerse Gat	1955	2.9	7 500	70
Haringvliet	1955	1.9	18 000	260
Brouwerhavense Gat	1955	2.4	30 000	325
Oosterschelde	1955	2.8	90 000	1 100

Table A4-1 Tide data of closures in the Netherlands

It was concluded that the works would be feasible, but that they so greatly exceeded the existing experience that only a phased approach could be successful, starting with the smaller inlets, and gradually tackling the larger inlets, based on experience gained during the project itself.

To reduce the current velocities during the final closure, it was advised that the discharge sluices should be built prior to the actual closures and that the capacity of the sluices should be used to divert some of the water in the final critical stages.

Finally, the Committee stressed the need to apply new technology and to carry out research by using all modern means.

The works proposed by the Committee have been indicated in Figure A4-1. The suggested time schedule anticipated a construction period of 25 years is given in Figure A4-2. Three major works determined the total time schedule: the closing of Veerse Gat, Brouwershavense Gat and Eastern Scheldt. Other elements, such as the closure of the Grevelingen, the Volkerak and the Haringvliet, had to be fitted in between.

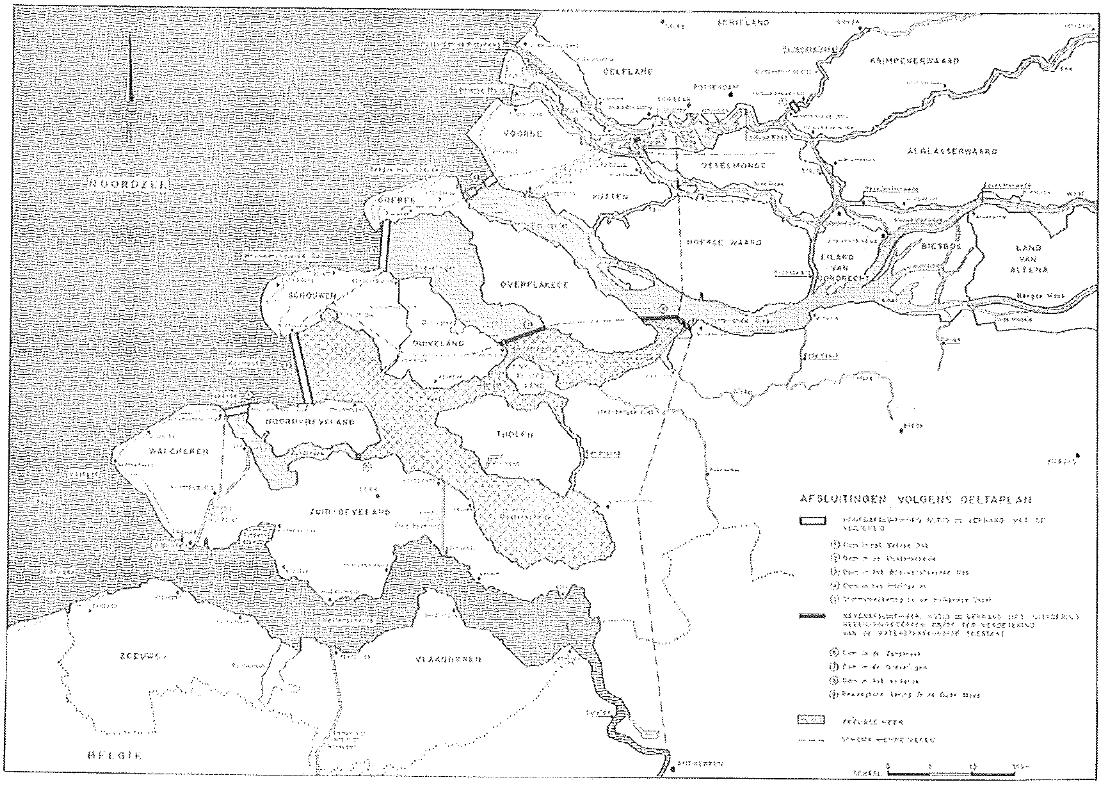


Figure A4-1 Delta area with major works

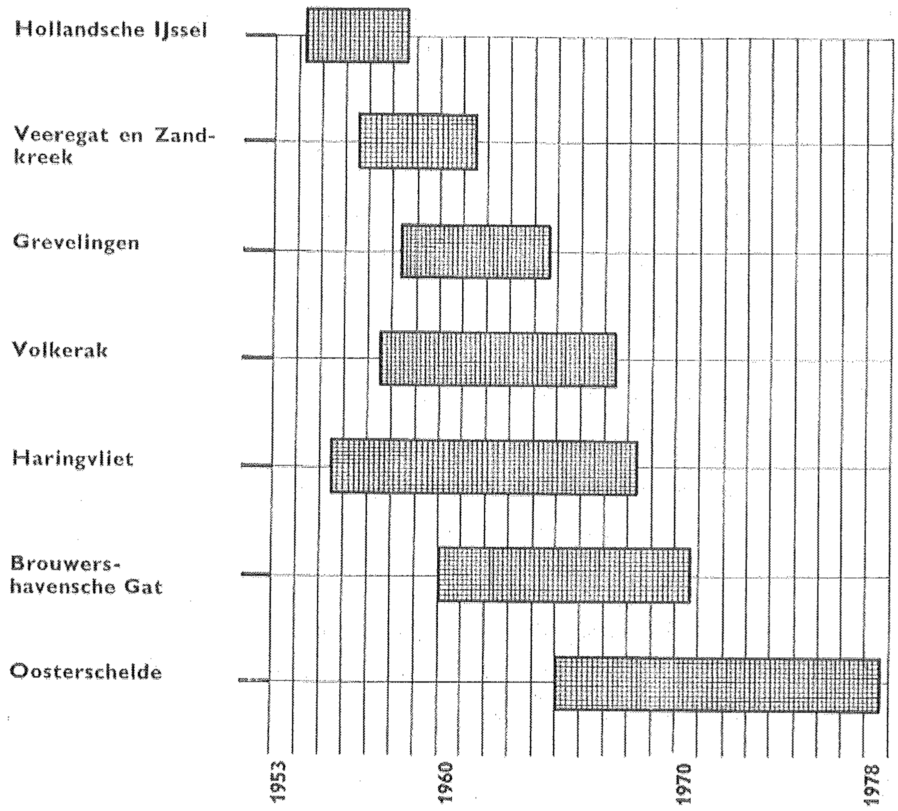


Figure A4-2 Time schedule construction Delta Works

A4.3 The execution of the works

For the management of the Delta project, a separate entity the Deltadienst within Rijkswaterstaat was formed:. Within this department, units were established for the design and supervision of the works, and for the research that had to be carried out to make the works feasible. For many years, the "Waterloopkundige Afdeling" guided the long term research, varying from field observations to all kinds of model tests and calculations.

The sequence of the works was partly designed to create a learning curve. Since it would be necessary to develop and test new techniques, a variety of working methods was used to provide experience and to make the appropriate choices for the most difficult closure -the Eastern Scheldt.

A good insight in the execution of the works can be obtained from the series "Driemaandelijkse Berichten Deltawerken" edited by the "Deltadienst".

Following the sequence of the works, highlights that are characteristic for each closure, along with the spin-off that was generated. will be mentioned .

- Storm surge barrier Hollandsche IJssel
  Flow profile 80 x 6.5 m; span of 80m very large in 1956
  Lock 120m x 24m

- Zandkreek
  Lock: 140m x 20m
  Closure: Unit caissons (closed type)

- Veerse Gat
  Seadike: extensive use of asphalt-concrete revetment in permanent structure
  Closure: first experience with (7) discharge caissons (doorlaatcaisson)
  First large scale application of geotextile for scour protection

- Grevelingen
  Lock 125m x 16m
  Closure:
    N. gap: experimental gradual vertical closure with cable cars
    S. gap: Unit caissons (closed type)

- Volkerak
  Dam from Over-Flakkee to Hellegatsplein: first experimental sand closures
  2 locks each 326m x 24m!
  Closure: (12) discharge caissons (doorlaatcaisson)

- Haringvliet
  Discharge sluice
    Constructed in situ in an artificial dock island 560 X 1400m
    Pile foundation
    17 openings of 56.5 m width each
    Visor gates supported by triangular (Nabla) girder
    Gates designed after extensive study of wave impact forces
    Sill and scour protection designed after extensive model testing
    Large scale use of asphalt and sand asphalt for slope protection
  Lock 124m x 16m
  Closure (with sluices open):
    N. gap: gradual vertical closure with cable cars
    S. gap: sand closure

- Brouwershavense Gat
  Closure:
    S. gap: gradual vertical closure with cable cars
    N. gap: (20) discharge caissons (doorlaatcaisson)

Until the closure of the Brouwershavense Gat, the time schedule of 1956 could be maintained with a slight delay of one year only. The hands-on learning process had resulted in spectacular improvements and innovations in the fields of:

- high capacity dredging;
- new techniques for scour protection, replacing costly and labour-intensive fabrication of willow mattresses (zinkstukken) by geotextiles and prefabricated asphalt mats;
- new techniques for revetments, replacing labour intensively placed blockstone (zetwerk) by asphalt-concrete;
- closure techniques, using open sluice caissons or, alternatively, gradual vertical closure.

In this way, in 1969, the formal decision to close the Eastern Scheldt could be taken. After careful consideration, the decision was taken to use the technique of gradual vertical closure by cable car. This method was preferred to the use of caissons to avoid the inherent risks of placing the caissons. The works on the Eastern Scheldt started with the construction of work harbours and dam sections over the shoals. A huge factory was constructed for pre-fabricated geotextile mats to be used as scour protection. The works were well underway, but in the public opinion there was a growing concern about the environmental effects of the large stagnant water basins. The potential loss of the oyster beds near Yerseke also continued to cause concern. Closure of the Eastern Scheldt even became an issue in the parliamentary elections of 1973. After the formation of a new coalition cabinet, it was decided to re-consider the continuation of the Delta Project.

Again, a committee (named after its chairman Klaasesz) was charged to find a solution. Early in 1974, the Klaasesz Committee came up with a compromise. Instead of closing the Eastern Scheldt completely, it suggested that a storm surge barrier be built in the alignment of the works that had already started. During normal tidal conditions this barrier would reduce the tidal amplitude to a level that was considered adequate for the oyster cultures. During storms the barrier could be closed to provide the desired safety. In the upper reaches of the estuary, some compartment dams would be built to separate the Rhine-Scheldt canal from the now tidal Eastern Scheldt, and to reduce high current velocities in the Krammer. Although the new plan was far more costly than the old one, the idea was received favourably for political reasons.

In its advice, the Klaasesz Committee recommended urgent further studies into the technical feasibility of the compromise. A group of experts from Rijkswaterstaat, the research institutes and the contractor of the works in the Eastern Scheldt, DOS, then carried out preliminary studies. On the basis of a six week study, the conclusion was that a storm surge barrier consisting of permanent sluice gate caissons would be feasible. It was then formally decided to suspend the current works and to start preparation of the works according to the new plan. Further studies were still going on to determine the exact location of the compartment dams in the upper reaches. The final layout is shown in Figure A4-3. Strict conditions were imposed by the government with respect to cost, time of completion and technical feasibility.

Preparation of the new design was not a task of Rijkswaterstaat alone, a design team was established with strong participation of the contractor. During the design phase, serious problems were encountered that forced the designers to a make radical change several times. First, the idea of using the caissons was abandoned to make place for a design with piers cast in situ and a foundation deep in the Pleistocene deposits, with the aid of cellular rings. Construction of the piers and their foundation would take place in separate steel cofferdams. This idea was also abandoned, and eventually the choice fell on prefabricated concrete piers that would be placed on mattresses consisting of a granular filter.

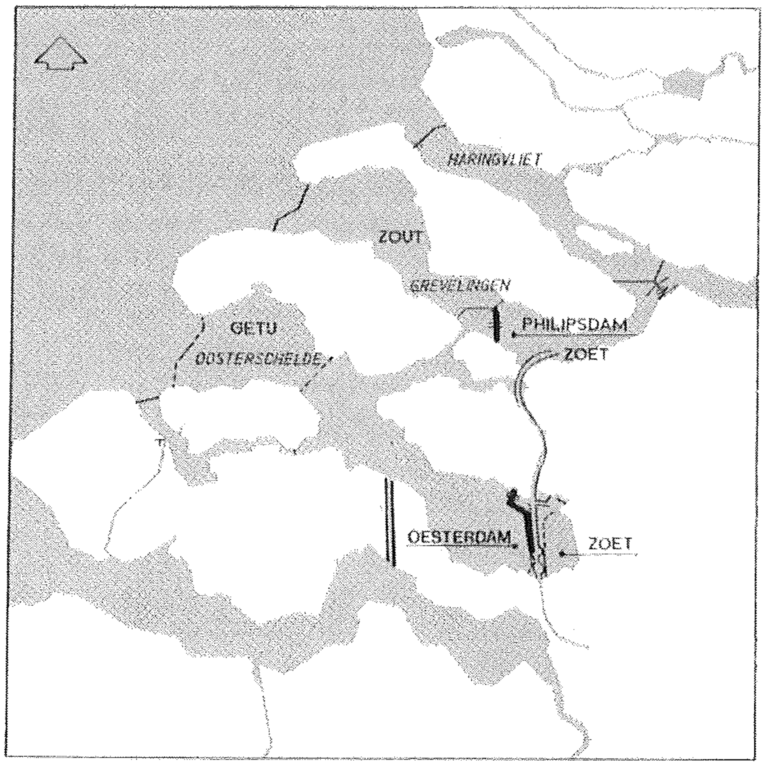


Figure A4-3 Compartment dams in Zeeland

In between the piers, a sill was to be constructed consisting of a sill beam and heavy quarry stone. The sill beam would be the lower support for the steel gates that would move in between the piers. In total, 66 piers had to be placed with a heart-to-heart distance of 45m, distributed over 3 main channels, thus forming 63 openings. The soil, consisting of loosely packed, fine sand had to be densified before the foundation mattresses could be placed. Accuracy was essential while working under extremely difficult conditions in water depths up to 35 m and current velocities of over 4 m/s.

Part of the already completed works had to be demolished including part of the scour protection and the piles for the cable way. The conditions set by the Government with respect to cost and time of completion were not met. The barrier was completed in 1986 instead of 1985, and the cost was also exceeded even after correction for the inflation. This could not be a real surprise since the original idea was abandoned to carefully build up experience during construction working from smaller to larger closures. The work methods required for the Eastern Scheldt barrier were a completely new challenge to the engineering community.

The works following from the revised plan are described briefly in the same way as for the other elements of the Delta Project:

- Eastern Scheldt
  - Foundation: compaction of sand by vibration to a depth of NAP - 60m
  - Placement of foundation mattresses consisting of 3 layers of granular material
  - Lifting and accurate positioning of piers in water depth of 35m and velocities up to 4m/s
  - Installation of sill and sill beams in high current velocities
  - Installation of (movable) gates between the prefabricated piers
  - Extensive scour protection works
- Philipsdam (Krammer)

Sand closure (due to manipulation with the gates of the Eastern Scheldt barrier to reduce current velocities

- Markiezaatskade
  Dam construction on very poor subsoil
  Closure: gradual horizontal closure using barges and trucks

- Oesterdam
  Sand closure (due to manipulation with Eastern Scheldt barrier to reduce current velocities

The works in the Eastern Scheldt added to the list of innovations in the following subjects:

- survey techniques
- accurate handling of very large and heavy concrete structures in extremely difficult conditions at sea
- probabilistic methods
- technology of granular filters
- scour and scour protection

A4.4 The Delta Project in hindsight

The Delta Project was originally designed in a period when awareness of the environment and of the ecological effects of civil engineering works scarcely existed. Moreover, the decisions to carry out the project were taken in an emotional context immediately after a major disaster that took over 18 00 lives. It is therefore not surprising that during the execution of the project priorities changed. This was strengthened by the growing level of prosperity and the growing attention for the quality of life.

The change in priorities that culminated in the re-design of the works in the Eastern Scheldt caused a lot of frustration amongst the staff involved. Certainly, the older people who had suffered during the economic crisis of the 1930's, during World War II, and finally during the inundation of 1953 could hardly understand that so much money was "thrown away" on a fancy item such as the environment. Even today, it is not clear whether the decisions of 1974, which were also taken in an emotional atmosphere, were wise ones. There has been no analysis to determine whether the money spent to "save" the Eastern Scheldt could not have been spent better (with a higher cost/benefit ratio) on other projects.

With our present appreciation of ecology and environment, we might doubt whether we should carry out the Delta project again if we had to face that question. We would certainly consider the loss of tidal wetlands and we would certainly consider the damming of the Haringvliet in relation to the large quantities of polluted sediment that are being deposited there. However, even today it is not easy to imagine what would have happened to those sediments without the Haringvliet closed. The sediments would have been discharged in an uncontrolled manner into the North Sea, and they would certainly have contaminated the Wadden Sea. In the absence of the Volkerak Dam, the same contaminated sediments would also have reached the Grevelingen and Eastern Scheldt basins that are relatively clean at present.

In conclusion one can say that the Delta project has certainly been very effective in reducing the risk of inundation. This aspect is gaining emphasis with the growing concern about the rise in sea level. Side effects of the project have been positive for the economic and social development of the region, including the opportunities for tourism and recreation. This is due not only to the better wet and dry traffic infrastructure, but also to the creation of the water basins. The effect on the national water management must be rated as positive. With the aid of the Haringvliet Sluice it has become possible to control the Rhine discharge, to reduce salt intrusion, and to safeguard drinking water resources.

Negative effects are the loss of tidal wetlands, the accumulation of polluted sediment in the Haringvliet and some problems relating to the water quality in the stagnant basins. However, with the aid of the hardware provided along with the Delta project, it is possible to counteract some of these negative effects. Finally, we must remember that any solution that could reduce the probability of inundation would have had negative effects. It is only for the solution that we have chosen that we really know what the negative affects have been.

Appendix 5 HYDROGRAPHIC CHARTS

A5.1 Introduction

Maps are a very important source of information for coastal engineers. This is true for both, maps of the land adjacent to the coast and charts of the seas and oceans. We expect that these maps will give accurate information about the topography of the area, but often additional information relating to land use, infrastructure, elevation, etc is provided. For the coastal engineer, charts of the seas and oceans are of particular interest. Such charts have been produced for many centuries to provide information to seafarers. The production of these charts was originally in the hands of private enterprises that had an interest in the trade between Europe and the East Indies and West Indies. In the early days of this trade, the maps and charts represented a great commercial value and they were kept secret by institutes like VOC and the British East India Company. Later, from around the start of the 19th century, with the formal establishment of the colonies the role of the governments in various countries became more important. The task of making proper maps of the sailing routes and the ports was then transferred to the various navies. Up to today, in most countries the national navy has a hydrographic department that is responsible for providing up to date information for the ocean navigation. An important part of that information is contained in hydrographic charts that give an impression of the local situation, including topography, bottom material, depths, sea levels, currents etc.

Such hydrographic charts are indispensable to sailors, and the presence of updated charts is mandatory on board of each seagoing vessel. When people cannot easily see what is below the surface of the water, maps and charts provide the only way for navigators to find out where it is safe for the ship to go and where it would be unwise to venture. Hydrographic charts are also an important tool for the coastal engineer, because these charts give reliable information on the conditions of the coastal zone. For engineers, however, it is not only the latest charts that are of interest, but certainly also the older maps that can still be obtained from the archives of the various hydrographic institutes. A sequence of maps gives a good impression of the long-term morphological developments. Historic maps can be obtained from the relevant hydrographic services.

This section shows the general principles governing the handling of maps and, more specifically, hydrographic charts and indicates roughly what information can be obtained from them.

A5.2 Units and their background

Hydrographic charts were meant to provide assistance to the navigators on board sailing vessels who had little more in the way of instruments than a clock and a sextant. Positions were determined with respect to the position of the sun and the stars. The grid of the hydrographic chart is therefore the grid of the degrees latitude and longitude as drawn on the globe. Transformation of this spherical grid to a plane map causes distortions, either in the centre or in the corners of the map. This means that the coordinates of the grid as indicated along the borders of the map are not linear.

Since the mutual distance between the longitudinal coordinates (meridians) varies (they are long at the equator and zero at the poles) only the degrees of latitude (parallels) give a proper indication of the scale of the map. The circumference of the earth is 40,000 km, which is divided into 360^0 (degrees), each consisting of 60' (minutes). This means that the 40,000 km are equal to 360*60= 21,600'. The sailors used the minute as their unit of distance, the nautical mile, which thus equals slightly more than 1850 m. The early hydrographic charts were not based on the metric system. Their scales therefore appear unusual to people who are familiar only with the metric system of measurement.

The speeds of vessels and the velocities of current are often expressed in nautical miles per hour (also called knots), which is slightly more than 0.5 m/s.

Water depth (soundings) are expressed either in the traditional nautical system, or in the metric system. This is always indicated on the map. The nautical system uses feet, fathoms (Dutch: vadem), or fathoms plus feet. A foot is equal to 0.3048 m; a fathom is equal to 6 feet or 1.83 m.

A5.3 Explanatory notes

The first thing to do when looking at a map is to study the key. The following information may be found:

- **Horizontal scale**. The scale indicating the dimensions of details at the map. Most sailors look at the latitude border scales at the side of the map. 1 Minute equals 1 nautical mile. Most coastal engineers look at the linear scale. As a result of the projection of the globe on a plane, the scale changes over the map. The larger the area covered by the chart, the larger the deviations.

- **Vertical Scale**. Depth on an admiralty chart may be shown in metres, feet or in fathoms. The reference level (Chart datum) of the depths is also important. Chart Datum is often related to some specified tidal data. This may be LAT (Lowest astronomical tide) or MSL (Mean Sea Level) or any other reference level. Different countries use different definitions of Chart Datum! A Belgian map of the Western Scheldt may thus give different depth values than a Dutch map of the same area. Even on one map, the CD may differ for different locations because the tidal data differ from place to place. This poses a risk for the coastal engineer since we usually assume that the datum level of a map is horizontal. Specifically when we make hydraulic calculations, we must make sure that we use levels that find a reference to a horizontal plane to eliminate errors in the gravitational forces. A striking example is given in Figure A5-1. The result of misinterpretation is given in Figure A5-2.

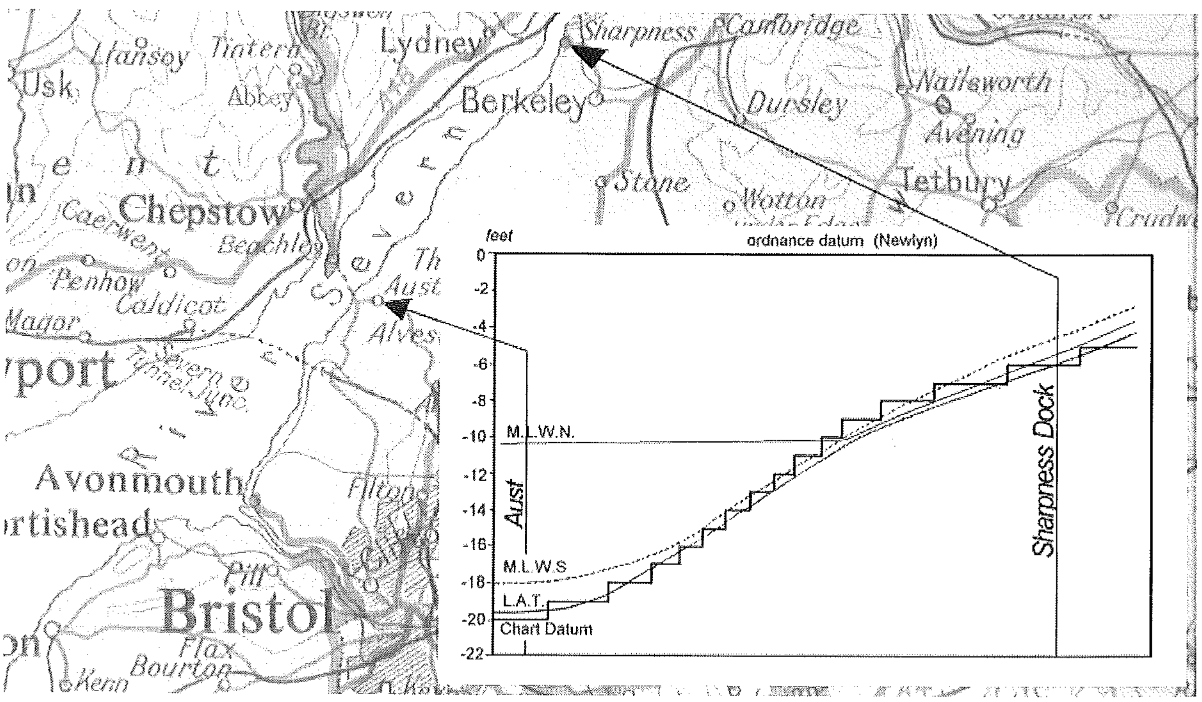


Figure A5-1 River Severn (England)

Figure A5-2 Chart Datum is not Horizontal!

- What happens to levels above the reference level like mud flats or sandbanks? These levels are underlined and refer to Chart Datum. Levels at the shore may also be shown. For these levels usually different reference levels are used rather than Chart Datum,
- Tidal levels are usually shown at some specific locations as shown in Table A5-1.

Place	Lat. N/S	Long. E/W	Heights in meters/feet above datum				Datum of Remarks
			MHWS	MHWN	MLWN	MLWS	
(position for which tidal levels are tabulated)							
			MHHW	MLHW	MHLW	MLLW	

Table A5-1 Tabular statement of semi-diurnal or diurnal tides

Terms related to tidal levels are summarised in Table A5-2.

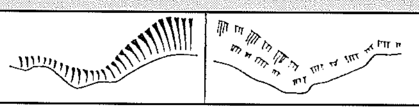
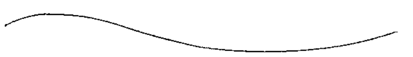
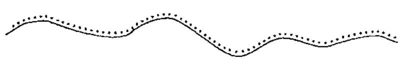
CD	*Chart Datum*
LAT	*Lowest Astronomical Tide*
HAT	*Highest Astronomical Tide*
MLW	*Mean Low Water*
MHW	*Mean High Water*
MSL	*Mean Sea Level*
MLWS	*Mean Low Water Springs*
MHWS	*Mean High Water Springs*
MLWN	*Mean Low Water Neaps*
MHWN	*Mean High Water Neaps*
MLLW	*Mean Lower Low Water*
MHHW	*Mean Higher High Water*
MHLW	*Mean Higher Low Water*
MLHW	*Mean Lower High Water*
Sp	*Spring tide*
Np	*Neap tide*

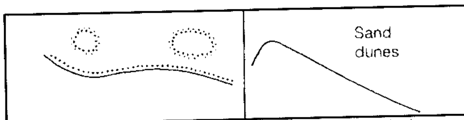
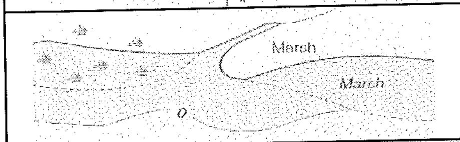
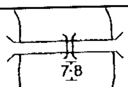
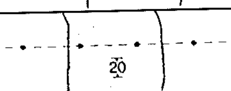
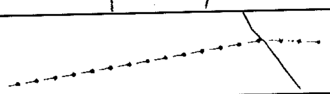
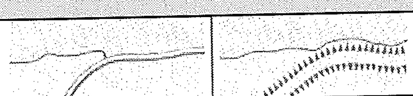
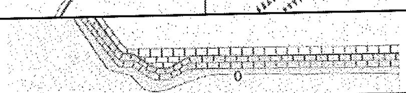
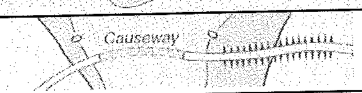
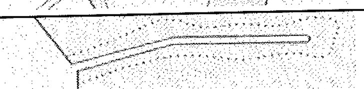
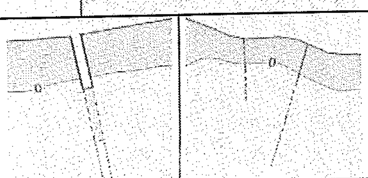
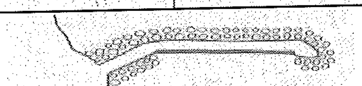
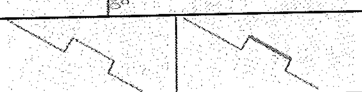
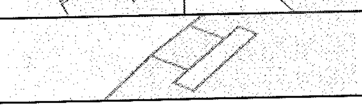
Table A5-2 Terms related to tidal levels

- Tidal streams/currents are sometimes shown.
- Dates of publication and dates of smaller or larger corrections.

A5.4 The map itself

Once familiar with this information the map itself may be studied. A summary of frequently used symbols is shown in Table A5-3. A complete list of symbols may be found in "Symbols and Abbreviations used on Admiralty Charts".

Natural features	
	Steep coast, cliffs
	Flat coast
	Sandy shore
	Stony shore, shingle shore

	Dunes
	Mangrove
	Swamp, salt marsh
Cultural Features	
	Buildings
	Bridges
	Cables
	Pipelines
Landmarks	
	Examples of landmarks
Artificial Features	
	Dykes
	Seawall
	Causeway
	Breakwater
	Groyne
	Mole
	Wharf
	Pier, jetty
	Pontoon

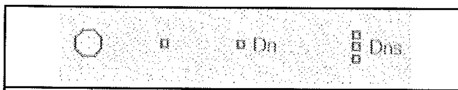
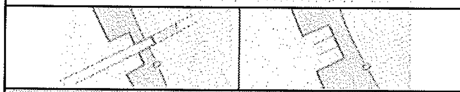
	Dolphins
	Ramp
Nature of the Sea bed	
S	Sand
M	Mud
Cy	Clay
Si	Silt
St	Stones
G	Gravel
P	Pebbles
Cb	Cobbles
R	Rock
Co	Coral
Sh	Shells

Table A5-3 Symbols and abbreviations used on admiralty charts

A5.5 Interpretation

Maps also may provide direct information about coastal processes like wind and wave directions and heavy breaking of waves. Many other phenomena can be derived by interpreting the coastal forms on the map: a spit indicates the direction of the longshore transport, and thus the dominant wave direction, and river sediment transport is shown by the presence of shoals and bars. If the map shows only a long straight sandy shore, you can say little about wave/wind direction and intensity. Only when there is some kind of interruption to this shore is it possible to determine the prevailing wave/wind direction and possible sediment transport. For example at a river mouth one may find out whether the river is dominant or the sea. The magnitude and direction of longshore sediment transport and river sediment transport may be deduced. Another example is formed by protruding rocks or artificial features like groynes or breakwaters on a sandy coast. Here also one may find indications of the presence of longshore sediment transport (magnitude and direction) and so the wind/wave direction. Detached obstacles (rocks, or detached breakwaters) may give even more exact information about wave direction. In the sheltered side sediment tends to settle. So the position of the shoals indicate the sheltered side and so the wave direction.

If dredged channels are present, it is clear that sediments have to be removed on a more or less regular basis. The location of the dumping grounds of dredged material give sometimes an indication of the dredging method that is commonly used.

In large sandy areas, the bottom contours also indicate possible current patterns. Places with great depths probably indicate areas where the current will be concentrated. Shallow areas indicate low current velocities.

A5.6 Limitations

Although hydrographic charts provide a very valuable contribution to our knowledge, the purpose of these charts is to assist navigators, rather than engineers. Soil data mentioned on the chart are generally only an indication of the surface of the seabed, they cannot be used in designing a foundation. The charts, and certainly the portions close to the shore, are meant to warn the sailors against running aground. Relatively more attention is therefore paid to shoals and low water conditions, rather than to gullies and extremely high water levels. Moreover, the scale of the charts is generally unsuitable for construction work. For the specification in tenders and project drawings, more detailed maps are required.

When comparing old and recent charts, one must be aware that the locations of buoys and lighthouses may have changed, since survey or the drawing of the original chart. Therefore, one should remain vigilant when using older maps for comparison. The most recent maps will probably be based on positioning with DGPS, an electronic positioning system using satellites as beacons. This eliminates most errors.

Appendix 6 THE CENTRIFUGAL PUMP

The centrifugal pump is the most common piece of equipment that is used to build up pressure in closed pipeline in order to overcome the pipeline resistance. In its simplest form the pump consists of a pump casing within which there is a rotating impeller (see Figure A6-1). The impeller is driven by means of an electric drive or a combustion engine.

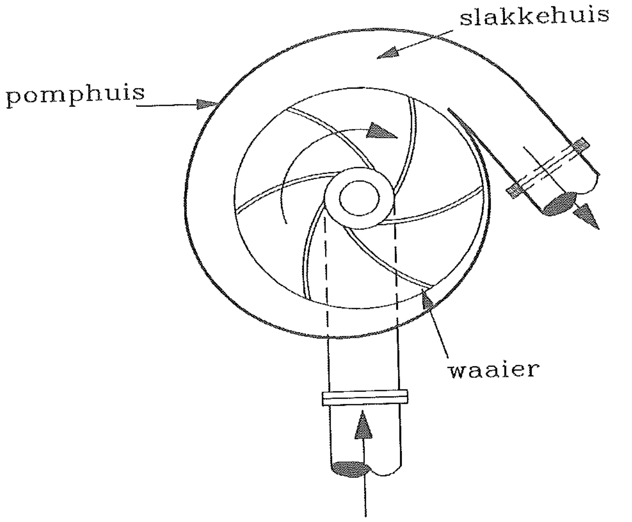


Figure A6-1 Cross-section of a centrifugal pump

The liquid that is to be pumped enters the pump casing via the centre of the impeller. Energy is imparted to the liquid by the blades of the rotating impeller. The liquid flows between the blades and thus arrives in the channel in the periphery of the pump casing and moves from this via the pressure side.

The flow in the pump can be considered to comprise the sum of two influences:

- the flow through the stationary impeller
- the rotation of the liquid in the impeller (see Figure A6-2)

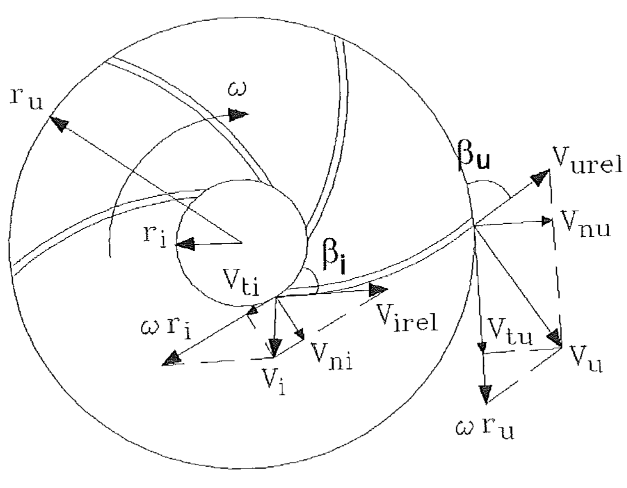


Figure A6-2 Rotation of the liquid

The inner circumference of the impeller is $2\pi r_i$ and the external circumference $2\pi r_u$. Because r_u is considerably bigger than r_i the tangential velocity of the liquid increases as it moves away from the centre. Through this mechanism, energy is transferred to the liquid. Velocity is transformed into pressure. It will be clear that the circumferential velocity of the impeller determines the head of the pump.

The liquid flows through the impeller at a velocity of v_{rel} the direction of which is the same as the direction of the impeller blades. The entry and exit angles are β_i and β_u respectively. Owing to the rotation of the impeller, the liquid also has a velocity ωr, in the tangential direction.

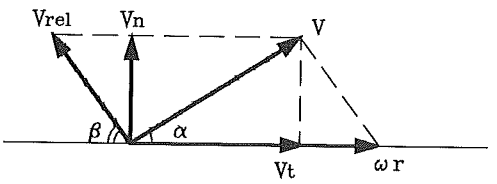


Figure A6-3 Velocities

The resulting movement of the liquid is found by combining the vectors v_{rel} and ωr. The characteristic picture is given in Figure A6-3. The resulting velocity is v. This resulting velocity can be decomposed into a component v_n, in the radial direction and v_t in tangential direction.

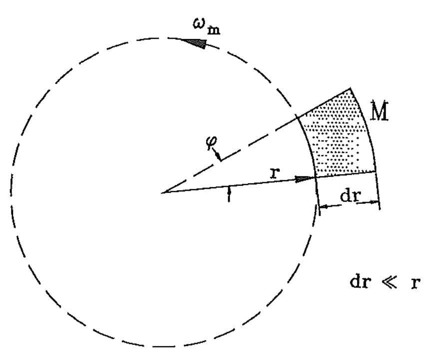


Figure A6-4 Rotating mass

The impulse balance between the inner diameter r_i and the outer diameter r_u can now be determined.

The impulse balance for a liquid particle that rotates round a centre is: (see Figure A6-4)

$$Tdt = d(J \cdot \omega_m) \tag{17.1}$$

In which
T = Impulse moment of the particle in relation to the centre
J = polar inertial moment of the particle in relation to the centre = $m \cong r^2$
m = mass of van the water particle
r = distance between the water particle and the centre
t = unit of time
ω_m = rotational velocity van the particle in relation to the centre

Now the impulse moment can be written as:

$$T = \frac{d(J \cdot \omega_m)}{dt} = \frac{d(\omega_m \cdot mr^2)}{dt} \tag{17.2}$$

The mass m is constant in time and amounts to $m = b \cong \rho \cong \varphi \cong r \cong dr$ (b = blade width). The rotational velocity is determined by the rotational speed of the impeller and the shape of the blades.

Thus:

$$\omega_m \cdot r \ = \ \omega \cdot r - V_{rel} \cdot cos(\beta) \rightarrow \omega_m \cdot r \ = \ V_t \tag{17.3}$$

in which:
ω = rotational velocity of the impeller
V_t = tangential component of the resultant velocity of the water particle

Thus:

$$\begin{aligned} T \ &= \ \frac{d(V_t \cdot mr)}{dt} \ = \ m \cdot \frac{d(V_t r)}{dt} + V_t r \frac{dm}{dt} \ = \\ 0 + V_t r \frac{dm}{dr}\frac{dr}{dt} \ &= \ V_t r (b \cdot \rho \cdot \varphi \cdot r) \frac{dr}{dt} \ = \\ V_t \cdot V_n \cdot b \cdot \rho \cdot \varphi \cdot r^2 \end{aligned} \tag{17.4}$$

in which:
V_n = radial component of the resultant velocity of the water particle

Taking the impulse moment of the water mass on the inner rim, we take $\varphi = 2\pi$ and $r = r_i$.

Now:

$$T_i \ = \ V_{ti} \cdot V_m \cdot b \cdot \rho \cdot 2\pi \cdot r_i^2 \ = \ \rho \cdot Q \cdot V_{ti} \cdot r_i \tag{17.5}$$

For the outer rim:

$$T_u = V_{ti} \cdot V_{ni} \cdot b \cdot \rho \cdot 2\pi \cdot r_u^2 = \rho \cdot Q \cdot V_{tu} \cdot r_u \tag{17.6}$$

The impulse balance is:

$$M_{th} = T_u - T_i \tag{17.7}$$

in which:
M_{th} = theoretical required external moment on the pump axis

Thus:

$$M \ = \ \rho \cdot Q \cdot V_{tu} \cdot r_u - \rho \cdot Q \cdot V_{ti} \cdot r_i \ = \ \rho \cdot Q \cdot (V_u \cdot cos(\alpha_u) \cdot r_u - V_i \cdot cos(\alpha_i) \cdot r_i) \tag{17.8}$$

The power that is exercised by the moment ($M_{th} \cong \omega$) is equal to the hydraulic power ($Q \cong \rho \cong g \cong \Delta H$).

From this follows:

$$Q \ = \ \frac{M_{th} \cdot \omega}{\rho \cdot g \cdot \Delta H} \ = \ \frac{\omega \cdot \rho \cdot Q \cdot (V_u \cdot cos(\alpha_u) \cdot r_u - V_i \cdot cos(\alpha_i) \cdot r_i)}{\rho \cdot g \cdot \Delta H} \tag{17.9}$$

in which:
ΔH= head [m]

From this follows:

$$\Delta H = \tfrac{1}{g}(\omega \cdot r_u \cdot V_u \cdot cos(\alpha_u) - \omega \cdot r_i \cdot V_i \cdot cos(\alpha_i)) \tag{17.10}$$

In addition:

$$V \cdot cos(\alpha) = V_t = \omega \cdot r - V_{rel} \cdot cos(\beta) \tag{17.11}$$

and

$$V_{rel} = \frac{V_n}{sin(\beta)} \tag{17.12}$$

and

$$V_n = \frac{Q}{2\pi \cdot r \cdot b} \tag{17.13}$$

Thus:

$$\begin{aligned}\Delta H &= \tfrac{1}{g}\{\omega \cdot r_u \cdot (\omega \cdot r_u - V_{urel.} \cdot cos(\beta_u)) - \omega \cdot r_i(\omega \cdot r_i - V_{irel} \cdot cos(\beta_i))\} \\ &= \tfrac{1}{g}\left\{\omega \cdot r_u\left(\omega \cdot r_u - \frac{Q \cdot cos(\beta_u)}{2\pi \cdot r_u \cdot b \cdot sin(\beta_u)}\right) - \omega \cdot r_i\left(\omega \cdot r_i - \frac{Q \cdot cos(\beta_i)}{2\pi \cdot r_i \cdot b \cdot sin(\beta_i)}\right)\right\}\end{aligned} \tag{17.14}$$

The values r_i, r_u, β_i, β_u and b are constant for a given pump. If the flow is stationary, ω is also constant. To simplify this the following constants can be inserted:

$$C_1 = \frac{(\omega \cdot r_u)^2 - (\omega \cdot r_i)^2}{g} \quad en \quad C_2 = \frac{\omega \cdot r_u \cdot cot(\beta_u)}{2\pi \cdot r_u \cdot b} + \frac{\omega \cdot r_i \cdot cot(\beta_i)}{2\pi \cdot r_i \cdot b} \tag{17.15}$$

then the relation between the head and the flow rate is:

$$\Delta H = C_1 - C_2 \cdot Q \tag{17.16}$$

This relation is shown in Figure A6-5.

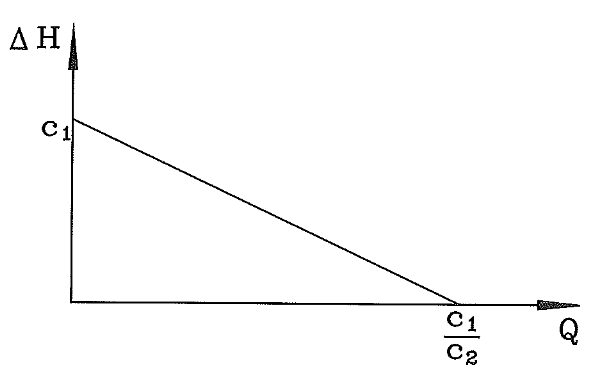


Figure A6-5 Theoretical *Q-H*-relation

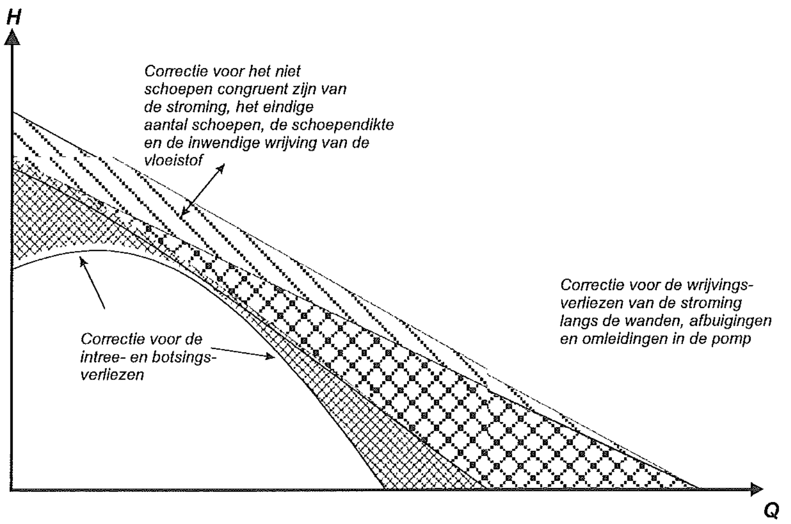


Figure A6-6 Actual *Q-H-curve*

If we transform the theoretical model to actual conditions for a centrifugal pump, it appears that it is necessary to make a number of corrections made to the theoretical curves (see Figure A6-6).

- Correction required because the because the flow rate is not congruent with the blades, the finite number of blades, places in a circular impeller, the blade thickness and the internal friction of the liquid.
- Correction for the friction losses along the walls, the bends and diversions in the pump entrance, the impeller and the pump casing. This correction increases with the increase in the flow rate through the pump. With continuously increasing flow rates this loss become increasing dominant until finally the total energy supplied to the pump is mainly lost in the intree- en stootverlies entrance and losses.
- Correction for the entry losses. For a given pump, the entry losses are only minimal at a single flow rate. Above and below this flow rate losses occur that increase with changes in the flow rate (higher of lower flow rates).

The pump characteristics

The characteristics of the pump are actually as shown in Figure A6-7. In this figure the manometric head H_{man}, the efficiency and power consumption of a pump for a specific speed is given as a function of the flow.

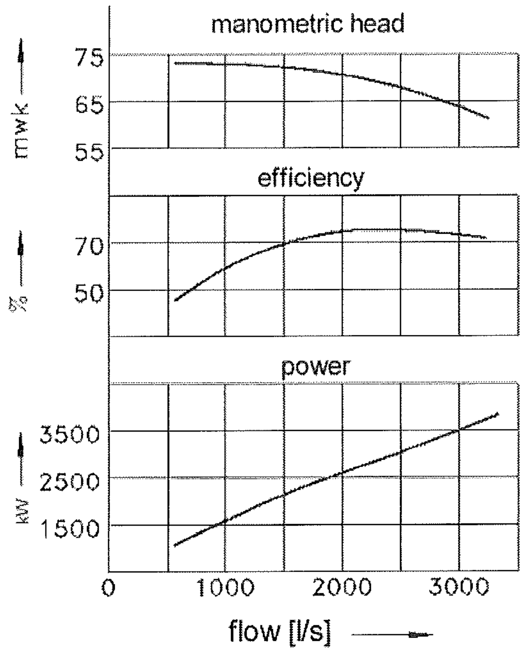


Figure A6-7 Pump characteristics

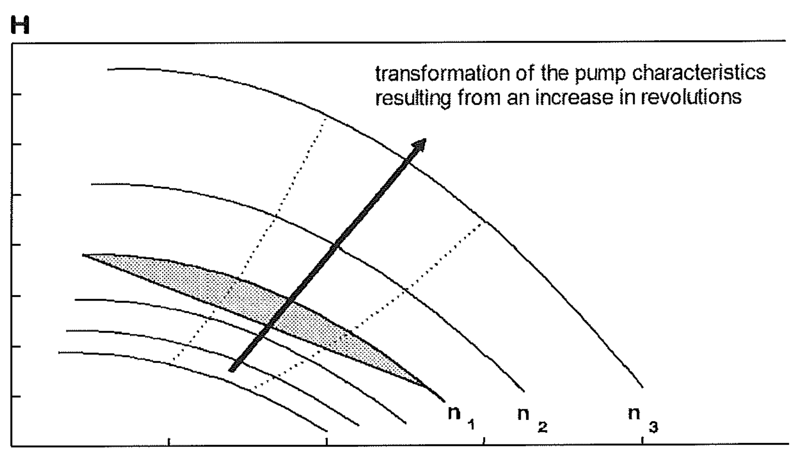


Figure A6-8 Actual *Q-H* curve

Figure A6-8 shows how the *Q-H* relation changes with changes in the speed (number of revolutions). If we have the *Q-H* curve of a pump and an efficiency curve for one revolution, it is possible to calculate the corresponding characteristics for a higher or lower revolution of the pump. If this relates to a relatively small change in the revolution, it can be assumed that the capacity and the manometric head will change in the following way.

1. The capacity Q is almost directly proportional to the revolution n, thus

$$\frac{Q_1}{Q_2} \therefore \frac{n_1}{n_2} \qquad (17.17)$$

2. The manometric head H_{man} almost directly proportional with the square of the exit velocity, thus:

$$\frac{H_1}{H_2} \therefore \left(\frac{n_1}{n_2}\right)^2 \tag{17.18}$$

3. The required power P is almost directly proportional with the cube of the revolutions, thus

$$\frac{P_1}{P_2} \therefore \left(\frac{n_1}{n_2}\right)^3 \tag{17.19}$$

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